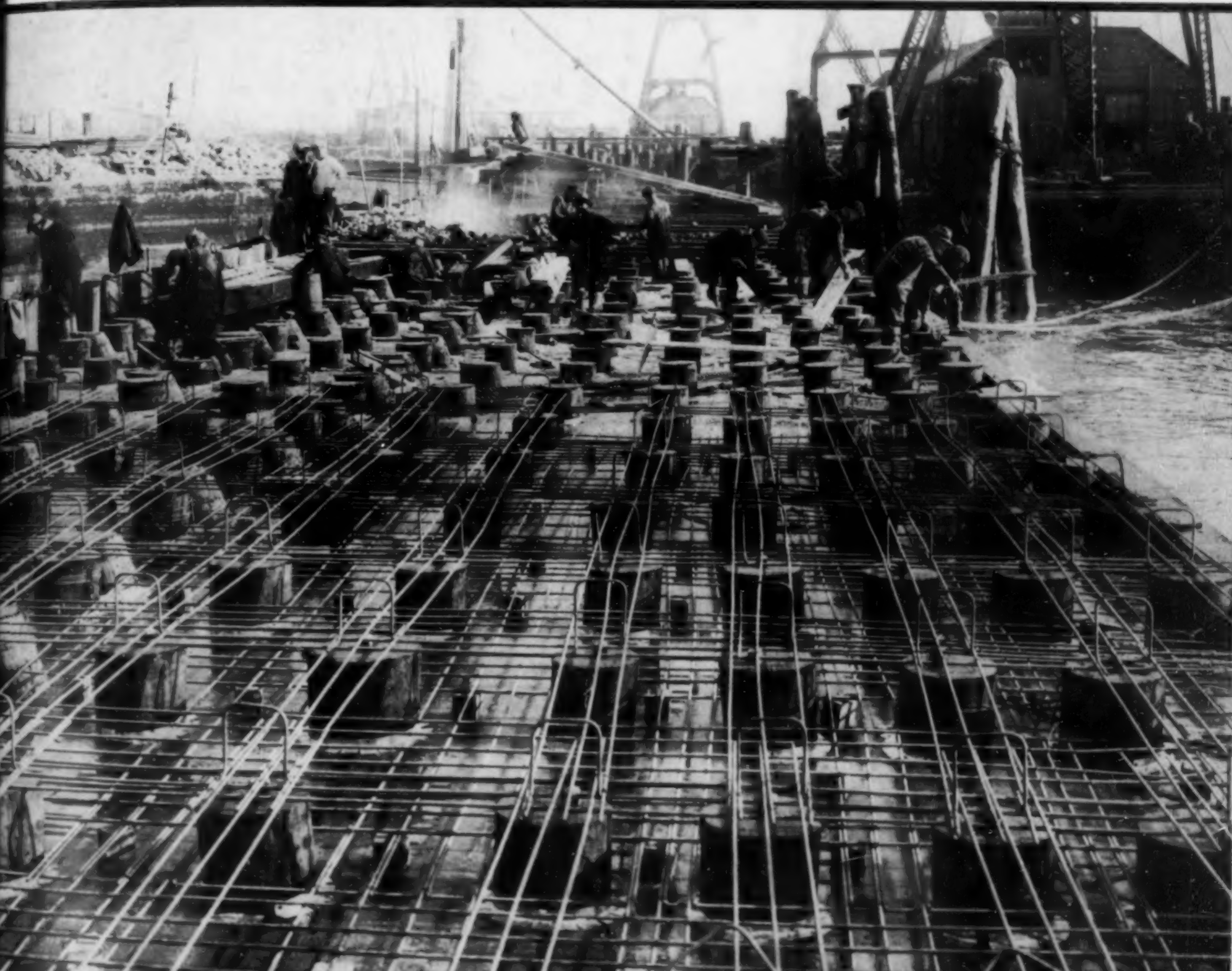
CIVIL ENGINEERING

Published by the American Society of Civil Engineers



RELIEVING PLATFORM BULKHEAD FOR NEW YORK'S EAST RIVER DRIVE, UNDER CONSTRUCTION
Steel Reinforcing in Place Ready for Pouring of Platform (See Article, Page 251)

Volume 12 — AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852 — *Number 5*

MAY 1942

Something to Think About

A Series of Reflective Comments Sponsored by the Committee on Publications

Cooperation for Stability

A Triple Alliance Within the Construction Industry Calls for Unity of Purpose

By F. A. Nikirk, M. Am. Soc. C.E.
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The construction industry is like a three-legged stool—it is supported by the engineer, the contractor, and the equipment manufacturer. Like the stool it requires the distribution of responsibility and mutual cooperation of its three supports.

All Are Interdependent.~The engineer needs the contractor, whose practical knowledge of organization and construction transforms plans into economic realities. The contractor needs proper equipment in order that he may complete his work expeditiously and at a cost commensurate with the benefits to be derived from the completed project.

A reliable equipment manufacturer must study construction methods and job requirements. He must design, test, and prove his products, and with proper management—of plant, machinery, labor, and supplies—produce construction equipment capable of doing the required work at a cost that will stimulate investment in other enterprises. Thus it is, through the combined efforts of the three component parts of the industry, that construction is encouraged and more work is developed for all.

One other bond between the three branches is the common approach and method of analysis required in the solution of their respective problems. The advantage of engineering training in this regard is evidenced by the increasing number of graduates from our engineering schools that are finding employment in all branches. While they thus have a common starting point, their subsequent experiences tend to pull them apart.

Must Maintain Contact.~Those going into contracting are immediately surrounded by practical problems where human contacts are most important. In the hustle and drive of a busy life they are often tempted to lose touch with engineering developments. If, however, they are ultimately to develop maximum efficiency, they should counteract this tendency through well-selected reading and membership in a technical society.

In the manufacturing branch, the engineer may be engaged in industrial research where he must be guided to a large extent by public demands and public acceptance, even though his work brings him very little direct contact with the public. If, however, he is engaged in sales engineering, he must keep constantly in touch with both science and the public if he is to attain the full measure of success.

Especially the Construction Industry.~The construction industry leaves to the engineering branch the liaison with scientific research, the interpretation of natural phenomena, and the application of the laws of nature to the design of structures. This involves the maximum of scientific study and the minimum of contact with society. The industry's analytical approach to problems is strange to the average layman. There is a tendency to develop a mode of expression that is unknown to the general public. In fact it may be said that they speak a different language from that spoken by their brothers with the same initial training who have gone into the other branches of the construction industry.

Any tendency to lose contact with the common life of mankind should be guarded against lest this should cause the technical engineer to withdraw into the temple of learning and science thus to become a priestcraft. In a democracy, the public is not inclined to trust its social and economic destinies to a clever sect.

One Would Help the Other.~Even a cursory view of the construction industry indicates the community of interest among the engineers engaged in its three branches and a vital need for cooperation among them. Such a conclusion should not stem from any fraternal instinct nor should it be accepted in an altruistic spirit. The material benefit would be sufficient to justify such cooperation. It would develop a healthy respect for the ability, integrity, and accomplishment of those working in allied fields and would render the experience gained in one branch available to others.

Of great value to the designing engineer is the experience of the constructing engineer. This experience determines the most economical method to be adopted in building a structure and the efficiency and limitations of the equipment chosen. Through the utilization of such experience and such knowledge, greater over-all economy in engineering work may be secured. Conversely, an understanding of the reasons why certain requirements or limitations are shown in the plans or specifications helps the constructing engineer in choosing

his methods to obtain the desired results. Such understanding likewise aids the equipment manufacturer in marshaling his experience and facilities for the production of a machine that will do the necessary work properly and economically.

College Training Suggested.—These principles are so fundamental, even so elemental, that it would seem unnecessary to mention them in this discussion. Experience, however, indicates that there is still much to be accomplished in this matter of cooperation. The mere willingness to cooperate on the part of engineers is not sufficient. The mere declaration of the desire or intention of cooperating on the part of those governing the policies of the various branches is not enough. The cooperation must be genuine and it must be active. It must inspire confidence and respect if it is to beget the cooperation of those in the other branches.

This situation seems to be improving as the number of engineers employed in all branches of the construction industry increases. To further this mutual understanding and at the same time enlarge the engineer's working knowledge, it might be well to consider including in the curricula of our engineering schools a course on the economics of selecting and using construction equipment. Such a course, dealing with the fundamental principles governing adaptability and performance, would afford an excellent opportunity for the practical application of engineering economics.

Many Engineering Accomplishments.—In spite of the need for more coordination within the industry, the engineer has had remarkable success in establishing the time-honored concept of engineering—that is, in controlling the forces of nature and directing them for the beneficial use of mankind. The construction of railroads, highways, airports, river and harbor works, bridges, pipe lines, and of dams for power, water supply, irrigation and flood control, has multiplied the national wealth. The development of power has made life much easier for the masses.

The telegraph, the telephone, the automobile, the motion picture, and the radio have combined to eliminate time and distance and have brought the layman face to face with facts and problems which were unknown to him two generations ago. They have been powerful factors in social and cultural advancement. They have helped to make comforts, pleasures, and even luxuries available to a greater percentage of the population than at any other period in history.

Selling Engineering to the Layman.—While all these achievements bear witness to the accomplishment of the material aims of engineering, they do not point to the achievement of those human values that are so important in a democratic society. Since the public is to be the final judge, it is quite as important to appear right as it is to be right. It therefore appears that the engineering profession has a selling job.

Democracy is founded upon the proposition that people are competent to determine their destiny. Engineering training teaches that competency to make decisions depends upon both the intellectual capacity and the opportunity to satisfactorily complete three preliminary steps—first, to obtain the necessary facts; second, to determine the significance of those facts; and third, to establish the laws governing the relationship between cause and effect. But the layman is seldom in possession of the necessary facts and his mind follows no such steps. When life becomes so complicated that he is unable to see, much less to understand, the forces working beneath the surface, he turns to someone for leadership and guidance.

Judged by Human Values.—A popular leader seldom follows the painstaking analytical steps of the engineer. He may even be an economic soothsayer who has no inhibitions about jumping to a conclusion and then amassing evidence to support his stand. We may expect him to belittle the material benefits accruing to mankind from the scientific advances of the past century and a half, and to blame the engineer for letting all the devils of technocracy out of Pandora's box to harass and annoy his fellow men. With the slightest encouragement he will pose as Epimetheus, promising to release that little creature called Hope, which will sooth the stings of technological inroads and bring about an economic Utopia.

If the construction industry and the engineering profession expect to obtain a favorable decision at the bar of public opinion, they must not rest their case on the material benefits rendered to mankind. They must pay more attention to human values than they have in the past.

Antagonism of Beliefs.—While those things which govern or control the existence and maintenance of life are of primary importance, it must be remembered that the beliefs and philosophies which define what is of value in the activities of life are often even more potent. The old axiom, "Self-preservation is the first law of nature," may be founded on sound psychology, but one may have cause to doubt it as he reviews history from the days of the early martyrs down to the present, when soap-box orators proclaim fantastic theories at the risk of personal violence.

In fact, some of the most violent disputes between individuals, groups, or even nations have arisen over differences in beliefs. These beliefs are seldom the results of independent analysis, but are often blindly accepted and followed because of the influence of heredity or environment. Politics, religion, free trade versus protective tariff, states rights, and many other issues furnish ample evidence of how bitterly such things are contested.

Cooperate to Mold Public Opinion.—This leads us to the inescapable conclusion that the engineer must give some attention to influencing public beliefs and philosophies. This means propaganda, a word which bears a stigma due to the perversion of its use. Perhaps a new word must be coined, but the fact remains that in a realistic world, propaganda is ever present. If those most vitally interested do not attempt to direct it, they should not complain of its adverse results in the hands of others.

If engineers will put forth a sincere effort to cooperate with others in the construction industry, they will not only increase their own knowledge and usefulness in their profession but will multiply their opportunities to guide public thought. They will then become, as Sir Clement Hindley says, "not only worthy members of a learned profession but also worthy citizens in a state whose affairs are influenced and directed by scientific truth."

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Volume 12

CIVIL ENGINEERING

May 1942

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Number 5

Parkways in New York Metropolitan Area

Observations on Large Mileage of Completed Four-Lane and Six-Lane Highways with Crossings at Grade Eliminated

By Sidney M. Shapiro, Assoc. M. Am. Soc. C.E.
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So efficient have become the parkway facilities around New York that the public, usually so hard to please, tends to take them for granted. But they have been developed only by dint of extensive study and vast expenditure. Mr. Shapiro briefly recounts something of the history, standards, and beneficial accomplishments, particularly in New York City and Long Island. This paper originated in the program of the Highway Division at the Society's recent Annual Meeting in New York.

To provide for rapid traffic movement around the entire circumference and through the interior of New York City is a tremendous task in itself. But in addition there are the needs for connections within the city and for major thoroughfares to and from the bordering communities of New Jersey on the west, Westchester County on the north, and Nassau and Suffolk counties in Long Island on the east (Fig. 1).

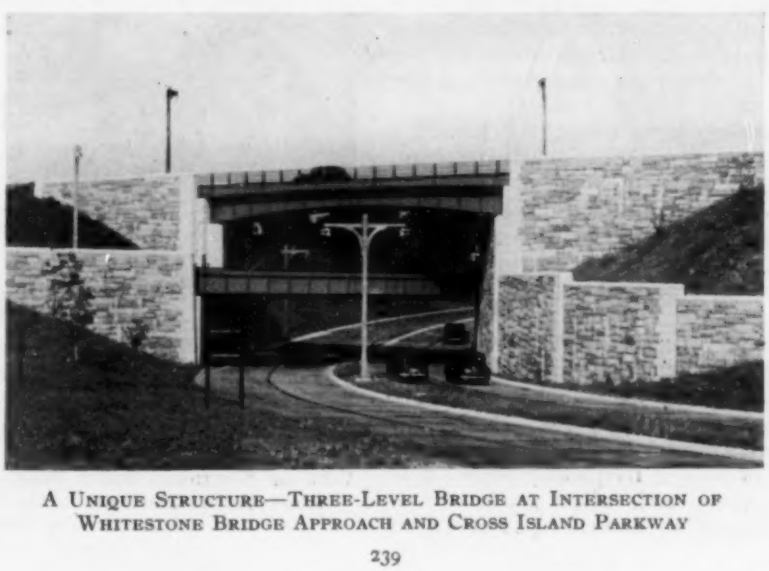
It all completes a great system of arteries, with major bridges and tunnels. In New York City and Long Island, the major development has been since 1926 under Commissioner Robert Moses; it consists of 164 miles of completed four-lane and six-lane roadways with all crossings at grade eliminated. A total of 280 grade separations have been erected to date. This article will deal only with the parkways and their incidental structures.

Construction consists generally of reinforced concrete pavement, with exits and entrances only at specified and regulated intervals. Roadsides are landscaped, billboards prohibited, and the latest features of traffic comfort and safety incorporated in the design. Bridges for the most part are reinforced concrete structures faced with stone. Included in the system are the five large bridges under the jurisdiction of the Triborough Bridge Authority (Fig. 1)—the Triborough Bridge, the Bronx-Whitestone Bridge, the Henry Hudson Bridge, the Marine Parkway Bridge, and the Cross Bay Boulevard Bridge. With the exception of these structures and their approaches, the entire system in New York City is under the jurisdiction of the New York City Department of Parks. The parkways on Long Island, outside the city limits, are under the jurisdiction of the Long Island State Park Commission; they were constructed in cooperation with the New York State Department of Public Works.

The parkways are reserved for the use of passenger cars and in this they differ basically from the conventional express highway or freeway. Highway officials throughout the country have already begun to press for segregation of passenger-car traffic, either by separate physically divided lanes, or by entirely separated roadways. The elimination of commercial traffic from an artery, reserving its entire capacity for passenger traffic, serves to make it more efficient and reduces the accident rate.

In the case of a parkway where a large demand exists for commercial traffic, provision is made for it by the construction of service roads immediately adjacent to the parkway proper. These service roads not only take care of all commercial traffic but are available for the use of local passenger traffic that would otherwise be forced to use the main parkway for relatively short distances. As a general rule, the service roads do not carry a sufficient load of traffic to warrant separation of grade at crossings but there are several cases in New York City where the service road along the parkway is provided with grade separation structures in the same manner as the parkway proper.

Entrance to and exit from a parkway are provided for only at planned intervals ranging from a half-mile to a mile in heavily built-up sections, and from



A Unique Structure—Three-Level Bridge at Intersection of Whitestone Bridge Approach and Cross Island Parkway

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COMPLICATED LONG ISLAND TRAFFIC WAYS—THE "KEW GARDENS PRETZEL"
Arteries Merge from Brooklyn (Upper Center), Triborough Bridge (Upper Right), and Long Island (Lower Left)

one to two miles in the open country. The fewer the exits and entrances the more continuous and steady will be the flow of traffic, with a resulting reduction in the number of accidents. An ideal situation, as far as reduction in accidents is concerned, would be one where the cars enter at one end of the parkway and leave at the other, with no intermediate entrances or exits. Since this is impractical, the next best solution is to have a minimum number of entrances to serve those motorists who wish to get on or off the parkway at various intermediate points.

RIGID RESTRICTIONS HAVE AIM OF MAXIMUM SAFETY AND ATTRACTIVENESS

Elimination of crossings at grade is a prime requisite of the parkway. This automatically obviates left turns, and it is the most important factor responsible for the increased capacity of a parkway as compared with the conventional highway.

Billboards and advertising signs along parkways are prohibited. In New York State no advertising signs within 500 ft of a parkway right of way can be erected except under certain conditions and with the approval of the parkway authorities. Here again lies an important, although indirect, economic advantage in the parkway—it invites traffic of the type that dislikes to ride on billboard-ridden highways.

Most New York parkway rights of way average 300 ft in width. There are occasions when through heavily built-up communities they have been narrowed to 200 ft, but this is fairly infrequent. On the other hand, through sections of open country, particularly through scenic areas, the right of way is frequently widened to as much as 400 ft. At cross roads where grade separation structures are necessary, additional right of way is essential for the proper design of access drives.

That part of the right of way of a parkway which is not utilized by the actual roadway or pedestrian walk is suitably landscaped. The area in the dividing mall and along the shoulders immediately adjacent to the paved roadway is generally reserved for lawn, and the rest is covered with suitable ground cover, shrubs, and trees. The parkway right of way, after proper landscaping, gives the effect of a "shoestring park."

DESIGN STANDARDS HIGH

The area adjoining the right of way in most cases is zoned for residential purposes. This not only protects the parkway from undesirable development adjacent to it but also enhances the value of privately owned property to the general advantage of the owners and of the community as a whole.

Certain basic features of design have generally been adopted for parkways. Divided pavement is standard design. The width of the center mall averages from 9 to 12 ft and the width of the concrete lanes is 11, 12, or 13 ft, depending on whether they are respectively inside, outside, or, in the case of the six-lane design, center lanes.

The concrete pavement design now in use calls for concrete colored with carbon black bordered by white-cement sloping curbs 3 in. high. The contrast between the black pavement and the white curb, as illustrated in one of the views, is of material help in outlining the edges of the roadway for night driving. At the same

VIEW OF NORTHERN STATE PARKWAY, NASSAU COUNTY, LONG ISLAND, SHOWS CONTRASTING COLOR OF DECELERATING LANES AND OF LOW WHITE CURBS

time, the dark pavement reduces glare during the daytime.

Funnel-shaped entrances to the parkway to prevent the simultaneous entrance of more than one car are also standard design. Experimentation in this design by several installations on the newer sections of the parkway has proved its value. This change in design has materially improved what was formerly a dangerous condition.

Accelerating and decelerating lanes are provided at each of the exits and entrances. These lanes are paved with uncolored (light) concrete in order to provide a color differential between them and the main through-roadway lanes. Alongside the parkway pavement the shoulder is graded level for a minimum distance of 8 ft. This is to provide space for disabled cars to pull off and make repairs on the grass outside the traveled lanes.

Lighting is provided by an improved incandescent system consisting of 400-cp lamps mounted on double-arm poles, located along the center mall and spaced 175 ft apart. The poles are of western red cedar, treated at the base, with a lamp mounting height of 23 ft. The center of the lamp is 6 ft out over the pavement from the pole. Sodium-vapor lighting has been installed on some of the newer parkway sections.

Bituminous macadam sidewalks, averaging 5 ft in width, are provided along the parkway for pedestrian use. Connections to intersecting streets permit convenient access to these walks. Benches are placed at suitable intervals.

Gas stations are constructed at selected points. They are designed of brick and stone to conform with the architectural treatment of the bridges. They average about $40,000 each in cost and are spaced approximately 10 miles apart throughout the system. State police facilities consist of main headquarters and barracks along the parkway, with substations and police booths at intervals equipped with police telephone and teletype systems.

As to grade crossing elimination structures, their design is predicated on a combination of factors. The relative advantages of over-crossings versus under-crossings, angle of skew, access drive arrangement, cost of land, architectural appearance, and other related items are all taken into account.

Completion of the comprehensive network of arterial parkways in the New York metropolitan area has resulted in certain definite economic benefits both to the motoring public and to the community in general. In the Borough of Queens, where only a decade ago traffic conditions were intolerable, the parkways have indirectly served to eliminate most of the traffic congestion on other thoroughfares and have provided rapid means of access for commuters to Manhattan and other parts of the city.

The same holds true in the neighborhood of the Henry Hudson Parkway where, in spite of the traffic induced by the new route, conditions on adjacent arteries

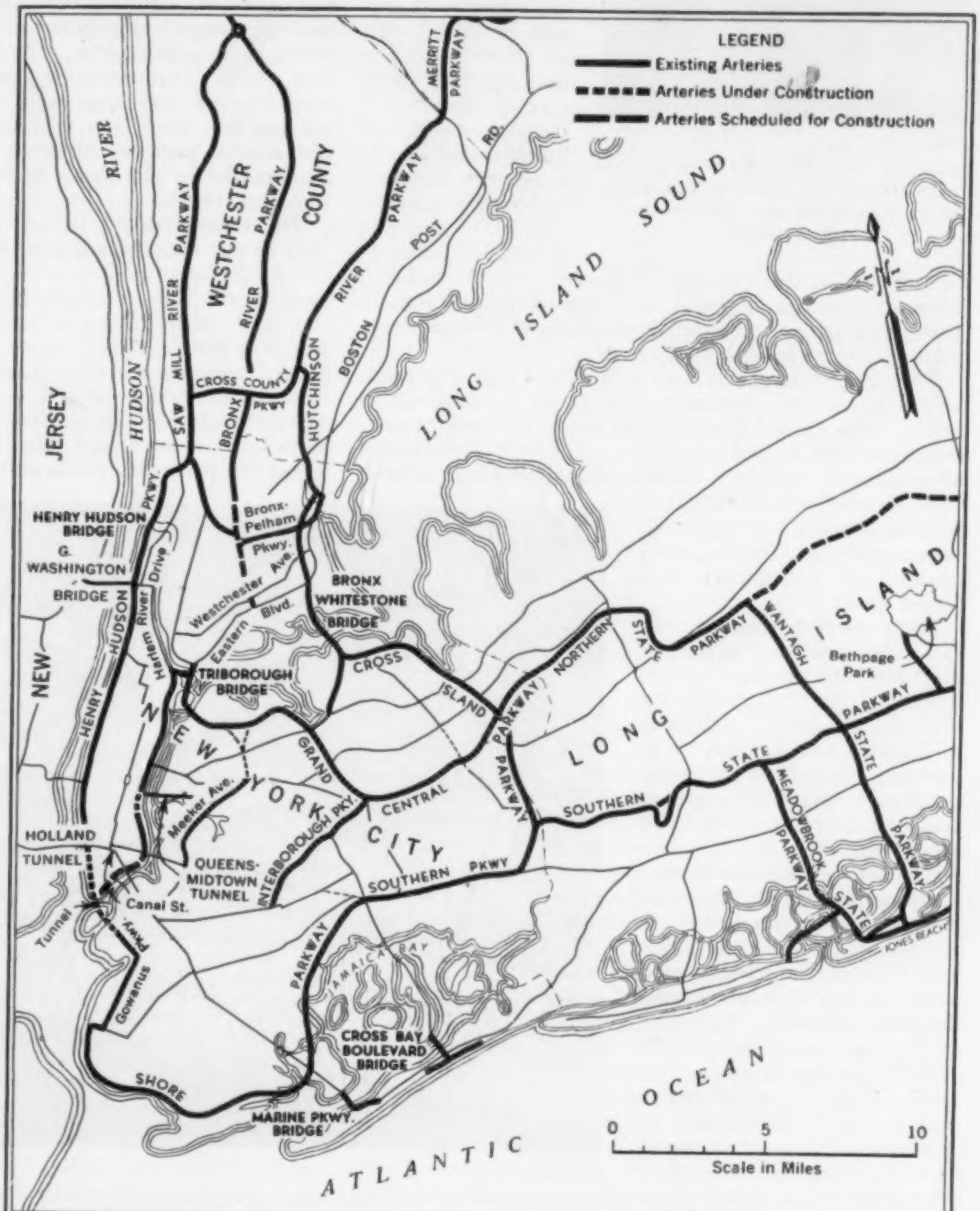

Fig. 1. Traffic Arteries Around Metropolitan New York

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SOUTHERN PARKWAY, QUEENS, ILLUSTRATES DESIGN FOR EXPRESS TRAFFIC IN CENTER ROADWAYS AND SERVICE ROADS EACH SIDE

have been improved. This is also true of the Hutchinson River Parkway Extension in the Bronx. Construction of a new parkway will affect the highways in the vicinity by removing from them a considerable amount of passenger-car travel, leaving them more usable for commercial traffic.

Statistics indicate that the average parkway designed according to standard practice, with all grade crossings eliminated, will carry approximately twice as much traffic as the average highway under the same conditions. There was a practical illustration of this in the Borough of Queens, New York City, where the Sunrise Highway, which until recently had no grade separations, ran parallel to the Grand Central Parkway, a fully grade-separated artery.

The Sunrise Highway had a four-lane concrete roadway 6 miles long, intersected by 13 cross roads, all at grade. The Grand Central Parkway is a four-lane concrete roadway, 7 miles long, intersected by 15 cross roads, all separated at grade. Each of these important arteries runs to the New York City–Nassau County line and each taps equally heavily congested centers of population and feeder roads.

Each carried traffic to the saturation point. The Sunrise Highway at that time carried 8,000,000 cars a year; and the Grand Central Parkway 15,000,000 cars a year. The carrying capacity of the grade-separated artery was almost twice that of the other. Besides, traffic on the 13 cross-roads intersecting the Sunrise Highway was badly throttled and congested, whereas the 15 roads crossing the Grand Central Parkway functioned normally, and without interference from the parkway.

The loss of time and capacity on the Sunrise Highway was economically so significant that funds were appropriated for grade separations; construction has now been completed by which this highway has been converted into a genuine parkway with no crossings at grade. This new section of parkway is a part of the Belt system.

Another important benefit of the parkway system lies in enchancement of the value of the surrounding property and the resulting increase in taxable values. It is a well-known fact that the increase in value in Westchester County and on Long Island by reason of the parkway system was tremendous, and there can be no question that these well-planned improvements are great assets to the county or city from civic, health, and esthetic viewpoints. These features tend to bring great numbers of people to the county and consequently to increase land values.

LOOKING BACKWARD

Speaking at a parkway opening recently, Commissioner Moses observed that in laying out the parkway program almost fifteen years ago there were no illusions as to the tremendous task ahead. The need for modern arteries of travel, including recreation facilities, had become a pressing, vital factor in the orderly development of the whole metropolitan area. To have delayed these necessary public improvements would have been a major civic blunder, and intolerable conditions would have developed had there been any further delay in advancing the arterial parkway program to meet the traffic problem created by the sharp increase in motor-car production and travel.

The Commissioner further stated that we are now well on our way toward a sensible solution of this problem in the metropolitan area. The completed parkway system; the Triborough Bridge, Whitestone, Henry Hudson, and other bridge projects; the new 35-mile Belt Parkway project in Brooklyn and Queens; and the many other improvements completed, under way, or planned, have established a pattern for an unparalleled arterial system made possible by the cooperation of the federal, state, city, county, and other authorities who have provided the necessary lands and funds for these projects.

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JONES BEACH STATE PARK ON THE OCEAN IS SERVED BY PARKWAY FROM MAINLAND

Huge Parking Lots in Middle Distance Give a Partial Measure of Traffic; Other Parking Areas Not Visible

Flood Control for the Yazoo Valley, Miss.

By George A. Morris, Assoc. M. Am. Soc. C.E.

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Development of a comprehensive plan for flood control in the Yazoo basin was complicated by the many interrelated streams, old and inadequate levee systems, and Mississippi River backwater. Extended studies led to the selection of four headwater reservoirs, supplemented by levees, cutoffs, and channel improvements. What led to this plan, how the reservoir and other work is being carried out, and what it promises to accomplish comprise the subject matter of this paper. In a compact form it gives an analysis of conditions throughout the valley, together with the solution of this major flood control problem.

Although studies for the final plan of flood control for the Yazoo River in Mississippi are still con-continuing, construction is proceeding rapidly on those features essential to all plans under consideration. The plan here described is tentatively based on four reservoirs and embodies the basic principles underlying any plan that may be adopted.

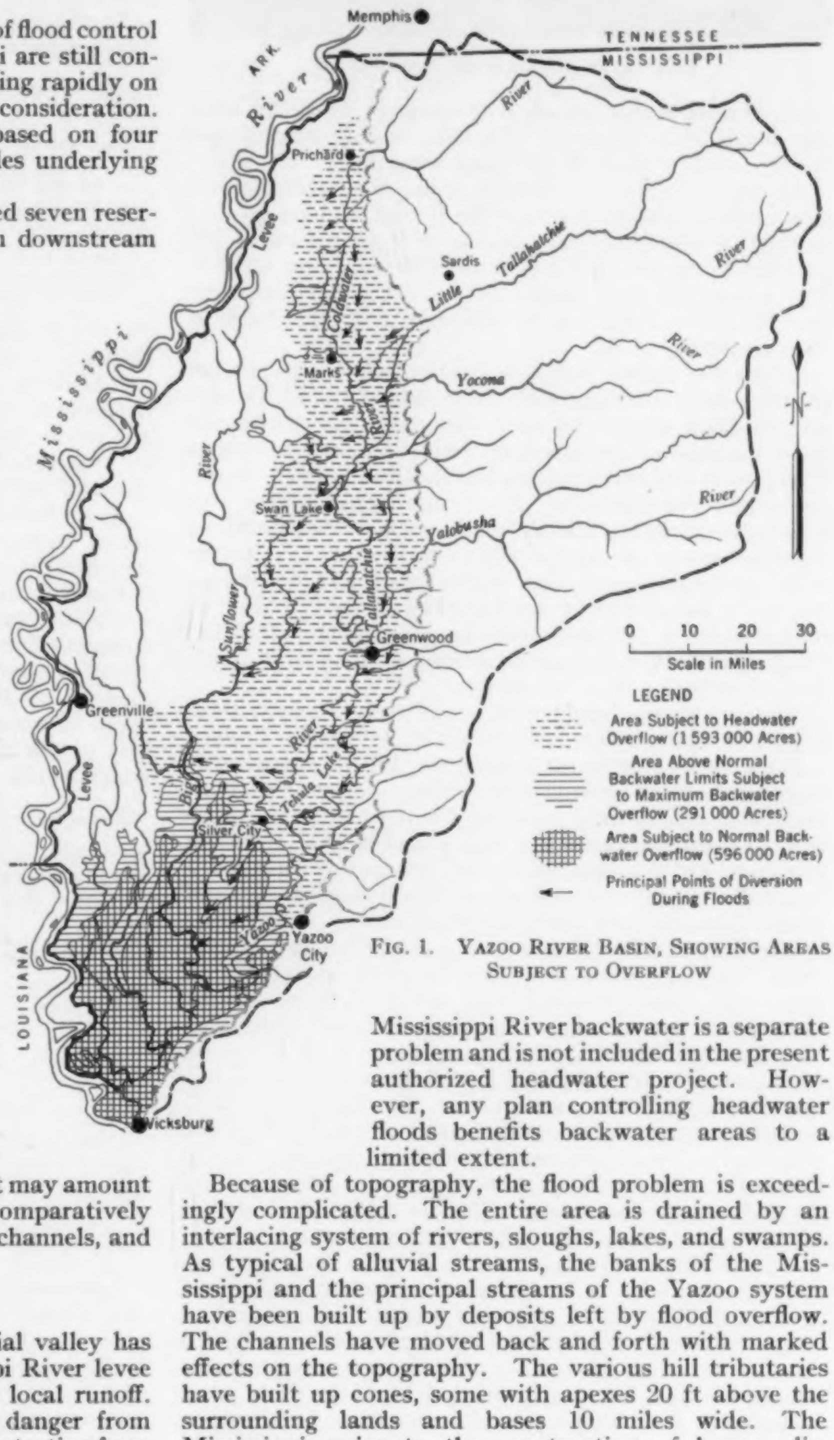


Fig. 1. Yazoo River Basin, Showing Areas Subject to Overflow

By legislation in 1936 Congress authorized seven reservoirs on hill tributaries, as well as certain downstream improvements; further it provided, "that the Chief of Engineers may, in his discretion, substitute levees, floodways, or auxiliary channels, or any or all of them, for any or all of the seven detention reservoirs."

Since 1936, investigations, studies, and designs have been made, all leading to development of this comprehensive plan. But because of the complexity of the problem, including unusual interdependence of local interests, all the features of the final plan have not been definitely determined.

The Yazoo River basin (Fig. 1) lies wholly within the State of Mississippi and comprises approximately the northwest quarter of the state. Its drainage area of 13,400 sq miles is about equally divided between rolling to rugged hill country and flat alluvial valley. Hill tributaries flow through valleys averaging one-half to two miles in width, with slopes averaging about 1.4 ft per mile. The main stream of the Yazoo-Tallahatchie-Coldwater system, following a meandering course throughout this entire alluvial valley, is characterized by stable, heavily wooded banks and flat water-surface slopes, which average only 0.26 ft per mile.

Climate in the valley is mild, the average annual temperature being 65 F. Most of the annual rainfall of 51 in. usually occurs from December to May. For the hill section, the storm runoff is ordinarily from 50 to 70% of the rainfall, but may amount to 80 or 90%. In the valley section it is comparatively low, averaging 25 to 35% with large lakes, channels, and swamps effectively regulating the flow.

A Real Flood Problem

In the past the Yazoo-Mississippi alluvial valley has been subjected to overflow from Mississippi River levee crevasses and to backwater, as well as to local runoff. With the enlargement of the levees, the danger from crevasses has been virtually eliminated. Protection from Mississippi River backwater is a separate problem and is not included in the present authorized headwater project. However, any plan controlling headwater floods benefits backwater areas to a limited extent.

Because of topography, the flood problem is exceedingly complicated. The entire area is drained by an interlacing system of rivers, sloughs, lakes, and swamps. As typical of alluvial streams, the banks of the Mississippi and the principal streams of the Yazoo system have been built up by deposits left by flood overflow. The channels have moved back and forth with marked effects on the topography. The various hill tributaries have built up cones, some with apexes 20 ft above the surrounding lands and bases 10 miles wide. The Mississippi, prior to the construction of levees, dis-

COMPLETED SPILLWAY OF SARDIS DAM

charged flood waters through openings in its banks, with the result that adjacent channels, such as Sunflower River, became much larger than normal.

At times of excessive rainfall, there is a natural diversion of flood waters through the banks. These go into storage in low-lying areas and ultimately return to the parent stream below or pass to a neighboring stream. The natural storage basins act as detention reservoirs and the rate of outflow is determined by the stage at their outlets.

When the valley was first settled, the fight against overflow from the Yazoo River system started. Gradually, natural diversions were closed by low dams and flood stages were increased, necessitating construction of levees, first by individual landowners and later by drainage and levee districts. No comprehensive plan was followed. Existing flood control works, constructed by local interests, are generally inadequate to provide effective protection. The valley area has undergone extensive development during the past 50 years, as indicated by a 250% increase in population. This has resulted in encroachment on low-lying areas and an increased need for flood control.

Existing channel capacities are entirely inadequate to pass even minor flood flows. The hill area, drained principally by four main tributaries (Fig. 1) is primarily responsible for flooding in the basin. Any one of these four is capable of far overtaxing the main system capacities. Runoff from only the valley area may be sufficient to cause local flooding but cannot produce general flooding. The extent of overflow and flood damage depends primarily on the location of the storm center. The period of overflow usually extends over a considerable period of time (Fig. 2). Flood flows, produced by initial storms, pass very slowly through the valley and are usually further delayed or augmented by additional storm runoff. In the alluvial valley region, some 1,593,000 acres (52% cleared) are subject to overflow from Yazoo River system floods alone, and about 900,000 acres (28% cleared) are subject to both Yazoo River flooding and Mississippi River backwater overflow (see Fig. 1 and Table I). Obviously any successful plan must control flood flows of the major hill tributaries by reservoir storage or by providing for their safe passage.

All possible flood control methods were therefore considered, singly or in combination. Such plans, for an agricultural area, must provide as nearly as possible a comparable degree of protection to all sections. Towns on relatively high ground are only flooded under extreme conditions and may be protected at little cost. Any lowering of flood stages cannot be localized but must be provided throughout river reaches of considerable length.

Studies indicated that reservoirs on the principal tributaries in the hill sections would be most effective for floods originating in those sections, and that their cost would be far less than that of levees and diversions. Further, such a reservoir scheme could be maintained and operated with greater certainty and safety than any other; it would be flexible and permit modifications as future conditions warrant. On the other hand, if the reservoirs were placed on tributaries near the mouth of the parent stream they would be more costly than methods, such as levees, for passing the flood flows safely through the comparatively short distance on the main stream. Reservoirs must be supplemented by levees and channel improvements to afford complete protection.

The topography of the valley is such as to divide it into a number of natural storage areas. Reservoirs on

TABLE I. FLOODS AND FLOOD DAMAGE ON THE YAZOO RIVER

Flood Period	General Location of Storm Center	Area Flooded (Acres)	Total Damage ($)
Dec. '31–Mar. '32	Middle basin*	1,139,000	1,353,000
Dec. '32–May '33	Middle basin*	577,000	863,000
Jan.–Apr. '35	Upper basin†	907,000	473,000
Dec. '36–Mar. '37	Upper basin†	769,000	200,000

* Middle basin includes watershed of Little Tallahatchie, Yocona, and Yalobusha rivers.

† Upper basin includes watershed of Coldwater River.

the hill streams, are in a measure, simply substitutes for natural valley storage. It is not feasible to provide sufficient reservoir storage or improved channel capacity to eliminate the necessity for natural storage areas. Thus in a sense these limited areas, acting as retarding basins, are essential features of the plan although they receive material benefits from the hill reservoirs.

As an indication of the value of natural valley storage areas, during the course of planning, careful consideration was given to a "delta retarding basin" on the Yalobusha River (Fig. 3). By constructing 24 miles of levee, some 900,000 acre-ft of storage could be obtained and complete flood regulation provided. Levees to provide 10-ft freeboard above maximum flood control pool would average 33 ft in height. The safety of the structure could be guaranteed by terminating the levee in such a way at the bluff line that before it could be overtopped water would pass around its end.

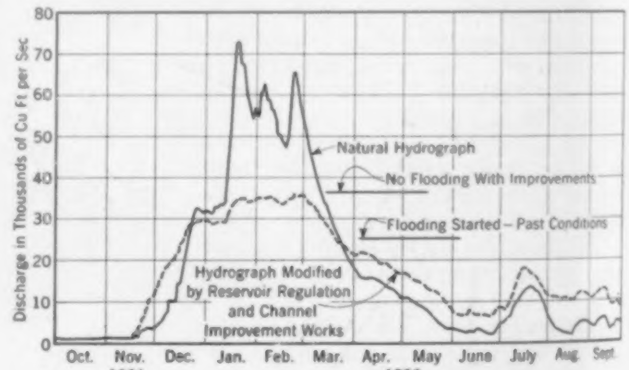


FIG. 2. COMPARATIVE DISCHARGE HYDROGRAPHS AT GREENWOOD, MISS.

Levees can be applied in whole or in part. They have the advantage of providing positive protection, available quickly without awaiting completion of a comprehensive reservoir system. Although levees cannot control floods greater than that used in their design, they will safely pass a series of floods of lesser magnitude, which if stored might exceed reservoir storage capacity.

To omit reservoirs and depend solely on levees in the Yazoo basin would require embankments aggregating 550 miles in length and expensive drainage works whose operation and maintenance would be burdensome. Levees parallel and adjacent to the bluff line are impracticable, because of expensive right-of-way requirements and maintenance difficulties occasioned by sediment-laden waters from numerous hill tributaries. Levees therefore must be limited to those areas where, either because of unusual topographic features or through the influence of the Mississippi River backwater, reservoirs and channel improvements cannot keep the flood flow lines below bankfull stage.

Channel capacities may be materially increased by clearing, enlargement, and straightening; however, it is not economical to carry such improvement beyond certain limits. In the Yazoo River basin where reservoir storage must be provided, to control a series of floods over several months, extensive channel improvement is of tremendous value in permitting increased reservoir ourflow and thereby decreasing storage requirements. Channel improvements usually are of very little value in reducing the cost of levee projects, where the flow carried by the river channel represents only a small proportion of the total flood flow. Channel improvements materially reducing natural valley storage must be undertaken very carefully, particularly during the period of construction of the comprehensive system.

Diversions in the form of leveed floodways or auxiliary channels are physically feasible in the Yazoo valley but are considered practical only where the flood flow, as modified by upstream reservoirs, cannot be safely carried by improved channels and low levees. Usually such diversions cause a deterioration of the original channel; however, this objectionable feature can be largely removed by providing regulating works at the head of the diversion to prevent flow out of the river except at maximum stages.

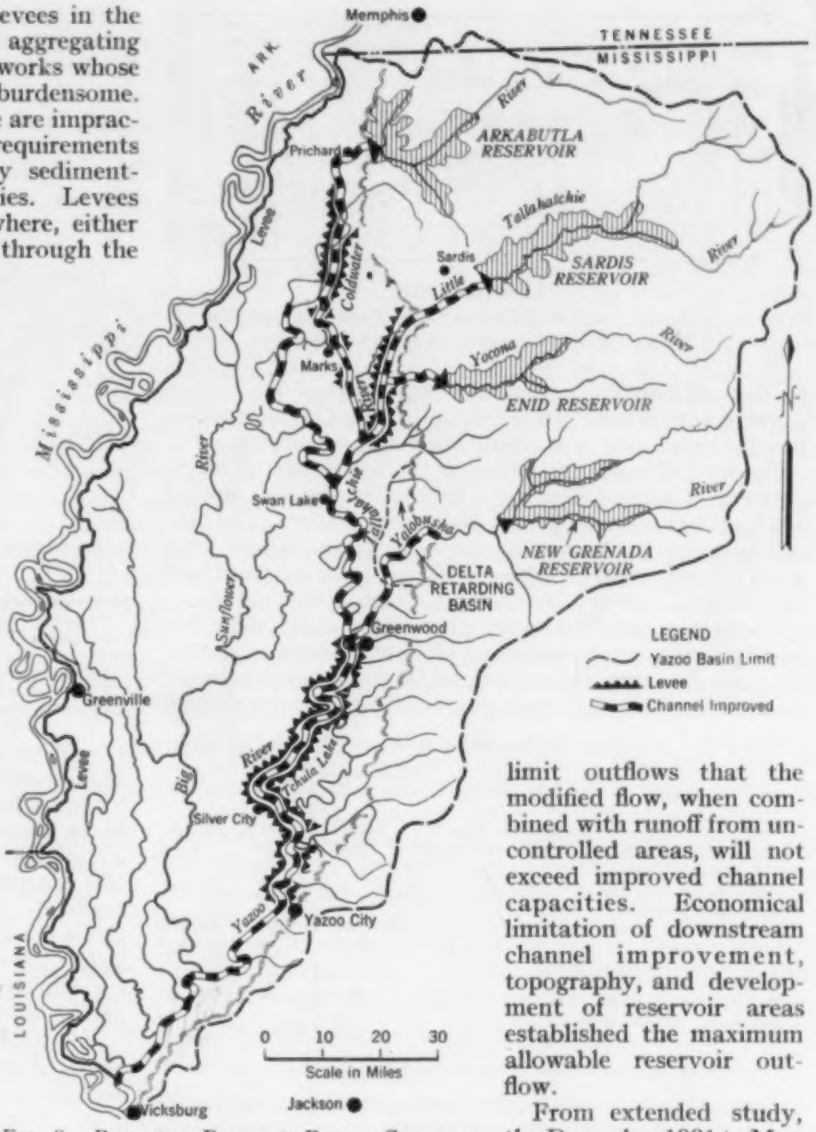


Fig. 3. Proposed Plan of Flood Control in Yazoo Basin

The problem of designing a plan of flood control for the Yazoo basin evolved into that of determining the correct balance between reservoirs, channel improvements, levees, and diversions. The essential features tentatively adopted are as follows:

1. A system of four reservoirs on the principal hill tributaries.

2. Improvement of main channels in the valley by channel clearing, channel enlargement, and cutoffs.

3. Low levees along certain reaches of the main streams.

Of the four reservoirs (Fig. 3), one is completed, one is under construction, and two others have been definitely located. Until recently consideration was being given to alternate reservoir sites on the Yalobusha River (Fig. 3).

Reservoirs included in this plan are Arkabutla on the Coldwater River, Sardis on the Little Tallahatchie, Enid on the Yocona, and New Granada on the Yalobusha. All are located in the hill section of the basin (Fig. 3). They will control 4,425 sq miles, or 67% of the total hill area contributing to headwater floods above Yazoo City. The general design features of all are similar. Flood control storage is designed to completely control the maximum storm series of record and to so limit outflows that the modified flow, when combined with runoff from uncontrolled areas, will not exceed improved channel capacities. Economical limitation of downstream channel improvement, topography, and development of reservoir areas established the maximum allowable reservoir outflow.

From extended study, the December 1931 to May 1932 storm series was found to be the most critical for flood-storage design. Reservoir gates are designed to control discharge during the flood season and to empty the reservoir during the low-water season. Outlets are designed to provide a discharge capacity at least twice the present planned outflow during the storing period, thereby permitting modification of operation in the event additional downstream works are constructed later. During the normal flood season, December through June, it is planned to maintain flow through a uniform number of gate passages and allow outflow to be regulated by variable heads of the flood control pool. During the normal low-water season, July through November, outflow will be increased to an amount which may be safely passed downstream. This method of operation was used for the design of the flood control plan; however, after completion of reservoirs and downstream improvements, more flexible reservoir operation may be dictated by experience.

At each site there will be an earth dam, constructed by rolled-fill or hydraulic-fill methods. Soils are similar in that all the valley bottoms have a relatively impervious top stratum varying from 5 to 50 ft in depth, underlain by sands and gravel, while the abutments are composed of sand, clay, and gravel blanketed by loess of varying thickness. The general design of dams will be similar

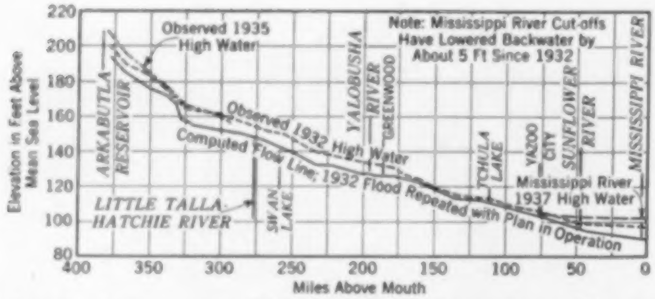


FIG. 4. COMPARATIVE WATER-SURFACE PROFILES ON YAZOO-TALLAHATCHIE-COLDWATER RIVER SYSTEM

to that of Sardis Dam (article by Norman R. Moore, CIVIL ENGINEERING, June 1939). Table II presents pertinent information concerning these hill reservoirs.

Existing channel capacities in the valley are to be increased. Channel clearing is limited to the area within top banks and includes some 500 miles of river. Channel enlargement is planned at those locations where local constrictions exist or where it is not possible to obtain the desired capacity by channel clearing and cutoffs.

Numerous cutoffs also are proposed. Since there has been practically no change in channel alinement during the past 30 years, channels must be constructed initially to sufficient size to obtain practically the entire desired cross section. A total of 30 cutoffs, having an aggregate length of 17 miles, will shorten the river 70 miles in a total of 312 miles (below Marks). Table III gives a comparison between existing and proposed channel capacities.

TABLE II. DATA ON RESERVOIRS FOR YAZOO FLOOD CONTROL

Item	Reservoirs: Arkabutla	Sardis	Enid	New Grenada*	Total
Drainage area, sq miles	1,000	1,545	560	1,320	4,425
Hill drainage area above Yazoo City, %	15	23	9	20	67
Flood control storage:					
Acre-ft	494,000	1,478,000	602,000	650,000	3,224,000
Inches of runoff	9.3	18.0	20.2	9.2	14.2 (Av.)
Maximum conduit outflow (cu ft per sec).					
During storing period	5,000	3,400	2,400	9,000	19,800
During emptying period	5,000	7,500	2,400	9,000	23,900

* Final determination of amount of storage to be provided has not been made.

Low levees are required along certain reaches of the main stream to prevent overflow of adjacent low-lying areas. Existing levees will be utilized where practical, supplemented by new levee construction relatively close to the banks, providing a batture of about 200 ft. Levee grades are based on a 3-ft freeboard above a flow line for the maximum flood of record, modified for reservoirs and channel improvements (Fig. 4). Wherever practical, spoil material from channel excavation will be utilized in the embankment. Drainage intercepted by levees will be cared for by flood gates and diversion ditches.

TABLE III. EXISTING AND PROPOSED CHANNEL CAPACITIES

Reach	Present Mileage* Mile to Mile	Existing Channel Capacity Cu Ft per Sec†	Proposed Channel Capacity Cu Ft per Sec†
Sunflower River to Silver City	44.4–117.3	25,000‡	32,000§
Silver City to Greenwood	117.3–185.1	17,000–25,000‡	32,000§
Greenwood to Swan Lake	185.1–257.6	11,000–18,000	20,000
Swan Lake to Little Tallahatchie River	257.6–274.8	7,000–11,000	9,000–17,000
Little Tallahatchie River to Marks	274.8–314.8	6,000	9,000 ‖
Marks to Prichard	314.8–365.6	2,500–5,000	9,000 ‖
Prichard to Arkabutla Dam	365.6–381.4	1,500	6,000

* Miles above mouth of Yazoo River.
† Corresponds to bankfull stage profile.
‡ With no Mississippi River backwater effect—below Sunflower River, Mississippi River backwater controls.
§ Channel at bankfull stage to be improved for 25,000–32,000 cu ft per sec or condition without Mississippi River backwater. Additional capacity provided by levees, which are required by reason of Mississippi River backwater.
‖ Additional capacity of 13,000–24,000 cu ft per sec provided by levees to care for short flood periods from minor tributaries.

OVER-ALL BENEFITS AND POLICIES

Under this plan, 1,350,000 acres, or 85% of the area subject to overflow from Yazoo River floods, will receive essentially complete protection, and 160,000 acres, or 10% partial protection. In addition, about 290,000 acres of the upper Mississippi River backwater area (Fig. 1) will be benefited to a limited extent. Table IV and Figs. 2 and 4 show the effects.

TABLE IV. PLAN OF FLOOD CONTROL—FLOOD DISCHARGE AND STAGE REDUCTION

Condition (For 1932 Flood)	Gaging Station: Swan Lake	Greenwood
Maximum flood flows (cu ft per sec):		
Observed	49,000*	73,000*
Modified	20,000†	37,000†
Maximum flood stages (ft above mean sea level).		
Observed	150.5*	132.0*
Modified	144.0†	126.4†

* Levee crevasses and natural diversions affected flood discharge and stage.
† With plan of flood control in operation—modified flood flow confined to improved channel.

Sardis Reservoir started in 1936, is completed. Arkabutla Reservoir will be placed in operation in 1942. About 300 miles of channels have been cleared, 26 cutoffs constructed, and 10 miles of channels enlarged and realined. In planning the construction program the following criteria have governed:

1. Elements must provide the greatest possible protection to the entire area without awaiting completion of the whole project.
2. Reservoirs on the hill streams should be constructed in order of the size of drainage area controlled and flood benefits provided.
3. Channel improvements should provide ample capacity for reservoir outflows during the emptying period without reducing natural valley storage.
4. They should start at the mouth and progress upstream, using the observed effects in designing additional works.
5. The entire plan should be flexible, allowing modifications or extensions as future conditions warrant.
6. Levees designed to pass flood flows regulated by reservoirs cannot be constructed until the required reservoir storage has been provided.
7. The program must utilize the total annual allotment of funds to the best advantage.

Observations to date show that the program is sound and that the basic designs and assumptions are conservative. Floods in the spring of 1940 were reduced an estimated 12 ft at Sardis on the Little Tallahatchie, and over 5 ft at Yazoo City on the main stream. To the present time, stages have not exceeded those observed in 1940.

Studies, planning, and construction of the entire project are under the direction of M. C. Tyler, M. Am. Soc. C.E., Brigadier General, Corps of Engineers, President, Mississippi River Commission, with Lt. Col. S. D. Sturgis, Jr., District Engineer, in immediate supervision. The writer was in charge of design and planning. His conclusions are not to be construed as necessarily representing the views of the U.S. Engineer Department.

Drop Structures for Erosion Control

East Bay Municipal Utility District, California, in Collaboration with Soil Conservation Service, Controls Erosion on Watershed Lands

By L. Standish Hall, M. Am. Soc. C.E.

Hydraulic Engineer, East Bay Municipal Utility District, Oakland, Calif.

On one California watershed, reservoir sedimentation surveys showed an erosion rate of 185 cu ft per acre annually. This condition called for serious consideration and the collaboration of the Soil Conservation Service. The structures used to meet the problem are here described in detail by Mr. Hall, and their behavior under heavy runoff is shown by numerous photographs. These, and the accompanying explanation, should be of considerable value to engineers interested in soil conservation on a watershed of this type. The article was originally presented as a discussion before the Hydraulics Division at the San Diego Convention of the Society.

For several years the East Bay Municipal Utility District has been interested in the control of erosion on its watershed lands in the vicinity of the East Bay cities of California in order to prolong the useful life of its local storage reservoirs. The District owns nearly two-thirds of the total watershed lands above its San Pablo, Upper San Leandro, Lafayette, and Chabot reservoirs, but rapid suburban development upon the lands privately owned has increased the rate of erosion during the past few years. Every highway, every railroad, the grazing of livestock, and the cultivation of lands—in other words, every man-made improvement which results in a scar on the earth's surface—introduces a real source of erosion that may reach serious proportions. Studies of the rate of silting of the District's local reservoirs indicated that the rate of erosion from these watersheds was one of the highest in the United States. (See the writer's article, "Silting of Reservoirs," *Journal of American Water Works Association*, January 1940.)

The rate of erosion on the San Pablo watershed, where conditions were the worst, was determined from reservoir sedimentation surveys to be equal to 185 cu ft per acre annually. Realizing this condition, the East Bay Municipal Utility District entered into a cooperative agreement with the Soil Conservation Service of the U.S. Department of Agriculture for erosion control on its watershed lands and in two years (since 1939) has constructed 20 drop structures of the type developed by the Soil Conservation Service for gully control following laboratory tests at the California Institute of Technology. Also 25 "Missouri-type" structure have been built. These consist of a pipe through the base of an earth embankment with a vertical riser at its upstream end. During the heavy storms of the 1939–1940, 1940–1941, and 1941–1942 winters, an opportunity was afforded to observe the performance of these structures under conditions approaching the maximum discharge for which the structures were designed.

A number of photographs were taken by the District's forces, to serve as a record of the performance of these structures. The accompanying photographs were taken from this series. A group of masonry drop structures is shown in Fig. 1 soon after construction work had been completed in the fall of 1939. These were built in a gully that was eroding very badly. Partly because of this erosion, and partly because of the construction work, there was very little vegetation left on the stream banks. On February 28, 1940, there occurred a downpour amounting to about 0.85 in. in a period of approximately 30 minutes. The resulting runoff over these structures was 81 cu ft per sec as compared with the design capacity of 88 cu ft per sec. The structures suffered no damage or erosion below them.

In Fig. 2 is shown the same group of structures one year later, photographed from a point slightly further downstream. It will be noted that vegetation on the banks has increased materially. Erosion from this drainage area has been so materially retarded that it now appears that several years will be required before the ponds above the check dams are completely filled,

Fig. 1. Group of Masonry Drop Structures Soon After Construction in Fall of 1939
Note Lack of Vegetation on Stream Banks

Fig. 2. Structures Shown in Fig. 1, a Year Later, from a Point a Little Further Downstream
Vegetation on Banks Has Increased

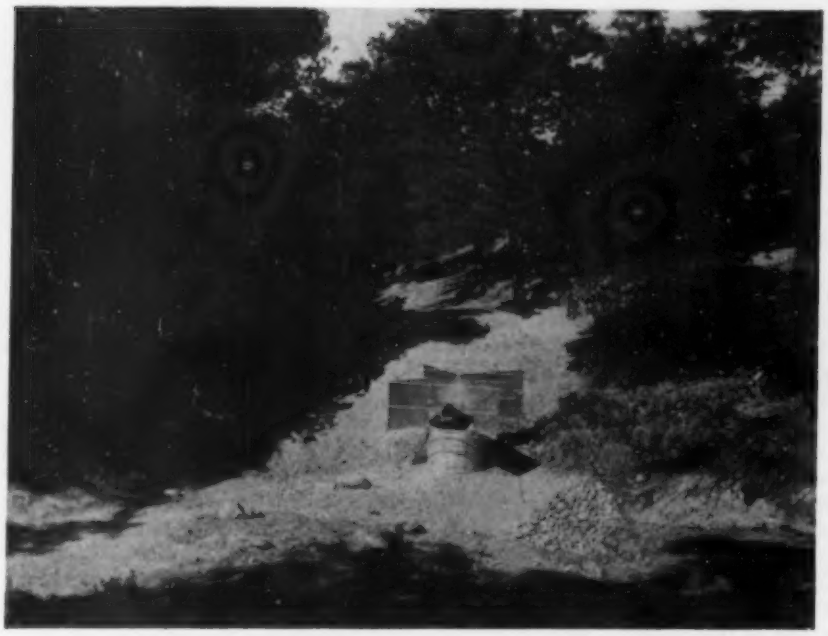


FIG. 3. DROP INLET OF CORRUGATED METAL PIPE TOPPED BY TRASH RACK; EARTH DAM JUST BEHIND

and the stream discharging from the last structure is practically free from silt.

Another type of control structure built was an earth dam (Missouri type) with a drop inlet of corrugated metal pipe (Fig. 3). The trash rack on top of the vertical riser has vertical vanes to eliminate vortex action of the water when the pipe is discharging under a maximum head. This idea originated with the Soil Conservation Service, but the design has been slightly modified by the East Bay District to conform to its needs.

This dam became filled with sand and gravel following a severe storm on April 4, 1941, when 0.76 in. in 30 minutes was recorded at the nearest rain gage. Judging from the volume of runoff, it is believed that the precipitation on the upper portion of the basin was much heavier than recorded. A bad slide developed near the head of the drainage area, which brought large quantities of sand and gravel mixed with mud into the pond above the debris dam, completely filling it. The trash rack was lifted off the drop inlet by a large tree which lodged in the cross bars and snapped the tie wires because of its buoyancy. The high-water mark was well above the top of the trash rack.

TRASH-RACK VANES SUPPRESS VORTEX ACTION

That such trash racks are effective in suppressing vortex action is shown by two views of different structures (Figs. 4 and 5). The first shows a 42-in. diameter drop inlet structure with the trash rack installed. The foam on the water surface has been carried into the pond by the inflowing stream and is not the result of vortex action. Figure 5 shows a drop inlet structure 36 in. in diameter. The trash rack has been removed and there is very pronounced vortex action. These two structures were both located on the same stream and the discharge at the time the pictures were taken was about 23 cu ft per sec at each. At maximum discharge, a vortex materially reduces the capacity of the pipe to carry the design flow.

A masonry drop inlet structure was located on the same stream as that shown in Fig. 3. The debris carried by the stream in the April 1941 flood passed through the outlet pipe of the earth dam (Fig. 3) and filled the pond above the drop structure (Fig. 6). These two structures restrained an appreciable volume of debris, estimated at 40,000 cu ft, from entering San Pablo Reservoir. Also they will reduce the rate of erosion from the lower part of this drainage area as vegetation becomes reestablished along the stream channel. The height of the drop is 7 ft 3 in. and the width of the notch is 17 ft 7 in. The depth of the pool is 1 ft. This structure has a designed capacity of 350 cu ft per sec.

Following the heavy downpour of April 1941, the observed depth of water on the crest of the weir was 1.92 ft and the discharge was about 200 cu ft per sec. The ratio of critical depth to height of fall was $dc/h = 0.18$. High-water marks disclosed that the peak discharge of 315 cu ft per sec resulted in a depth of 2.67 ft over the notch. At this discharge there was some turbulence below the pond, caused by the water impinging on an earth embankment on the right side of the pool which was at a greater height than the pool lip. Except for the removal of this projection during the flood, no bank erosion downstream from the structure resulted from the turbulence in the stilling pool.

The same structure after the conclusion of the 1940-1941 winter flood season appears in Fig. 7. The pond above the drop is filled with debris, and the oak trees lining the banks, which were formerly being undercut by the stream, are now well supported around the roots. Gravel was carried over the crest, filling the stilling basin. The channel cutting below the center of the sill is clearly evident in this view, but at the same time the absence of bank scour should be noted. The subsoil formation of the channel banks is a sandy clay. Some scour occurred in the center of the channel but in general the stream bed was the same as in Fig. 6. Prior to the erosion control program, large cobbles of andesite were deposited in the stream bed below the dam from an outcrop at the head of the drainage way over a mile distant. This gives an idea of the transporting power of the stream during floods.

The effectiveness of the longitudinal sills in preventing bank scour is indicated in Fig. 8, which shows the bank below the apron of one of the structures shown in Figs. 1 and 2. This bank had stood up under two seasons of unusually heavy runoff. The flow over the structure in February 1940 was practically equal to the design capacity.

The structure shown in Fig. 9 was completed during the fall of 1940. The total flow during the winter was very heavy, although the maximum discharge did not exceed 50 cu ft per sec. This view was taken from the top of the wall, and shows the pool and the longitudinal sills. The structure was designed in accordance with the formulas proposed by Morris and Johnson. (See "Hydraulic Design of Drop Structures for Gully Control," by B. T. Morris, Jun. Am. Soc. C.E., and D. C. Johnson, Assoc. M. Am. Soc. C.E., PROCEEDINGS, January 1942, p. 17.) The drop is 9 ft 3 in., the width of notch 6 ft, and the depth of pool 1 ft 1 in. The design capacity is 83 cu ft per sec. It should be noted that although the channel width contracts materially below the apron, there is no evidence of bank scour. The earth projecting above the slope of the masonry side wall has been scarcely disturbed.

One of the largest structures built by the East Bay District for erosion control is shown in Fig. 10. This

Fig. 4. Vanes on Trash Rack Above Drop Inlet Structure Suppress Vortex Action

Fig. 5. Vortex Action Around Drop Inlet Structure in Absence of Trash-Rack Vanes

Fig. 6. A Masonry Drop Structure Passing a Flow of 8 Cu Ft per Sec

Designed Capacity Is 350 Cu Ft per Sec

Fig. 7. Structure Shown in Fig. 6 at Conclusion of Winter Flood Season

Pond Above Has Become Filled with Debris

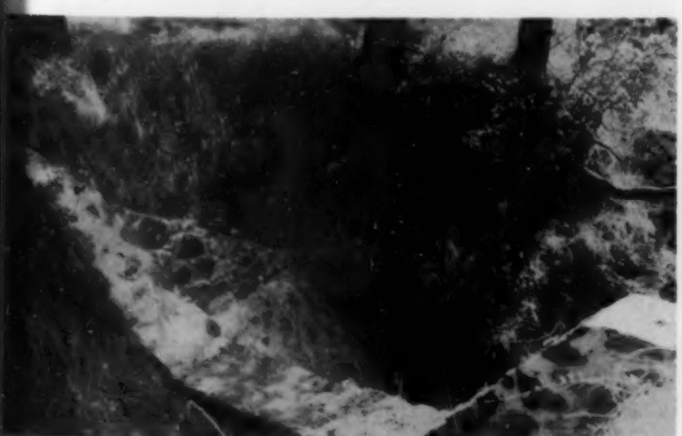


Fig. 8. Same Structure Shown in Figs. 1 and 2 Looking Downstream

Note Effect of Longitudinal Sills in Eliminating Bank Scour

Fig. 9. Close-up of Pool Below a Drop Structure After a Winter of Very Heavy Runoff

Contracting Channel Below Apron Has Not Caused Bank Scour

FIG. 10. CONCRETE DROP STRUCTURE WITH CAPACITY OF 6,000 CU FT PER SEC

Note Longitudinal Sills in Stilling Pool—Forms Not All Removed

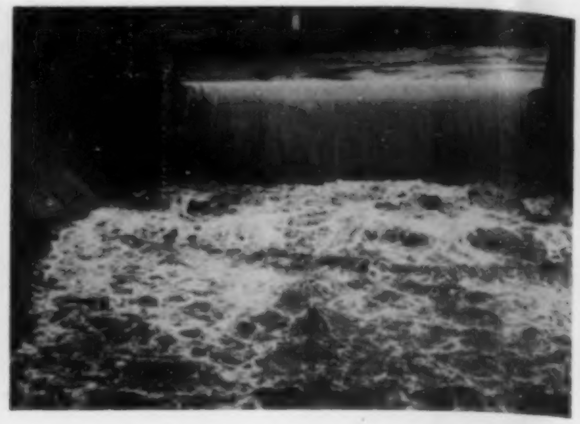


FIG. 11. STRUCTURE SHOWN IN FIG. 10 PASSING A FLOW OF 680 CU FT PER SEC

Benefit of Longitudinal Sills Clearly Indicated

concrete dam, of flat slab type, was built on San Pablo Creek a short distance above the San Pablo Reservoir. The spillway has a designed capacity of 6,000 cu ft per sec. The length of crest is 58 ft 5 in., the height of drop 13 ft 6 in., and the depth of pool 3 ft. This view, taken shortly after the construction had been completed and before the forms had been entirely removed, plainly shows the longitudinal sills in the stilling pool. This structure was designed before the results of the experiments at the California Institute of Technology were available, and therefore is of slightly different dimensions from those of the proposed formulas previously referred to. The length of the pool is 42.8 ft, as compared with a length of 30.6 ft calculated by the formula (Eq. 6 (*a*), *ibid*., p. 39). The depth of pool is 3.0 ft, as compared with a calculated depth of 3.45 ft. The longitudinal sills are placed a distance from the side of the notch equal to 0.333 of the crest length rather than 0.275, as required by the formula (Eq. 14, *ibid*., p. 45).

At a discharge of 150 cu ft per sec over this dam, the depth of water on the crest was 0.8 ft, giving a ratio of $dc/h = 0.04$. The damping of the turbulence in the pool was very effective with this small flow. Two larger discharges over this weir crest are shown in Figs. 11 and 12.

In Fig. 11 the discharge is 680 cu ft per sec with a depth of 2.3 ft over the crest, giving a ratio of $dc/h = 0.11$. The end contractions on the weir crest should be noted and the whirlpools created in the pool as the discharge deflected toward the center of these contractions impinges on the longitudinal sills. The small turbulence at the downstream edge of the pool and at the sides of the pond should also be noted. This view clearly indicates the benefit of the longitudinal sills.

A discharge of 1,020 cu ft per sec gave a depth of water over the weir crest of 2.95 ft and a ratio of $dc/h = 0.15$. This flow appeared similar to that shown in Fig. 11 except that the turbulence and whirlpools in the basin below the dam increased in intensity.

The final view (Fig. 12) shows the maximum discharge over the dam during the 1941 runoff season. The flow at this time was 1,800 cu ft per sec with a depth of water over the dam of 4.2 ft and a ratio of $dc/h = 0.21$. Note the pronounced end contractions on the weir and the ropey character of the discharge. The water at this time was carrying a very large volume of suspended matter over the dam. The turbulence in the pool has increased, although it may still be noted that there is a marked reduction in the turbulence below the downstream edge of the pool. The performance of this structure during the 1940–1941 and 1941–1942 seasons was very satisfactory. Although the basin pool dissipated the energy of the falling water, a sack concrete toe wall with willow bank protection was required on the right bank, and a post, wire, and brush revetment was required on the left bank at this location, because of the curvature of the channel below the drop structure.

As an erosion-control structure, its effectiveness to date has been in removal of the coarse bed load in the stream. The pond above the dam is not of sufficient length to thoroughly remove the finely suspended matter, and much of this passes over the crest of the dam during large floods. The runoff on this drainage area is very flashy and the flood flows only continue for a period of a few hours after the precipitation has ceased. The filling of the stream channel above the dam with sediment will within the next year or two greatly increase the vegetative growth and help to stabilize the stream gradient on the portion of the watershed above this structure.

The research work carried on by the Soil Conservation Service to determine the proper hydraulic design for drop structures will prove of great value to all hydraulic engineers. The erosion control program instituted by the Service for the conservation of soil on agricultural lands has been given a new application in the control of silting of storage reservoirs, and offers a possible means of prolonging the life of these important structures.

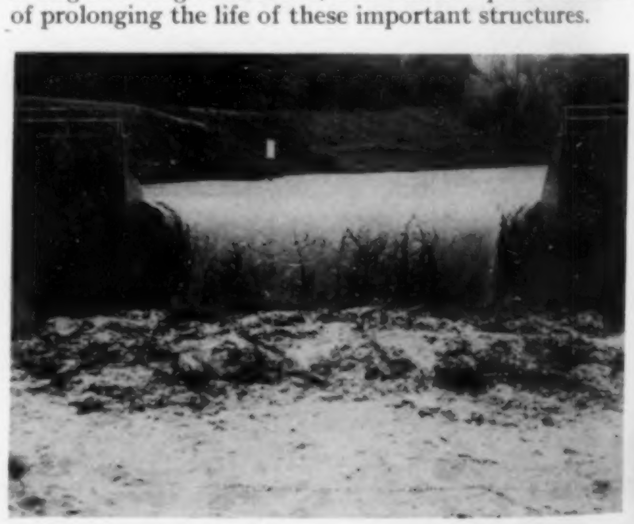


FIG. 12. STRUCTURE SHOWN IN FIGS. 10 AND 11 PASSING 1,800 CU FT PER SEC, MAXIMUM DISCHARGE OF 1941 SEASON

Note the Marked Reduction in Turbulence Below the Downstream Edge of the Pool

Sanitation Dump Building and 3,000-Ton Coal Pockets Next to Bulkhead Line near Queensboro Bridge

Relieving Platform Bulkhead for New York's East River Drive

By Lester C. Hammond, M. Am. Soc. C.E.
Chief Engineer, Borough of Manhattan, New York, N.Y.

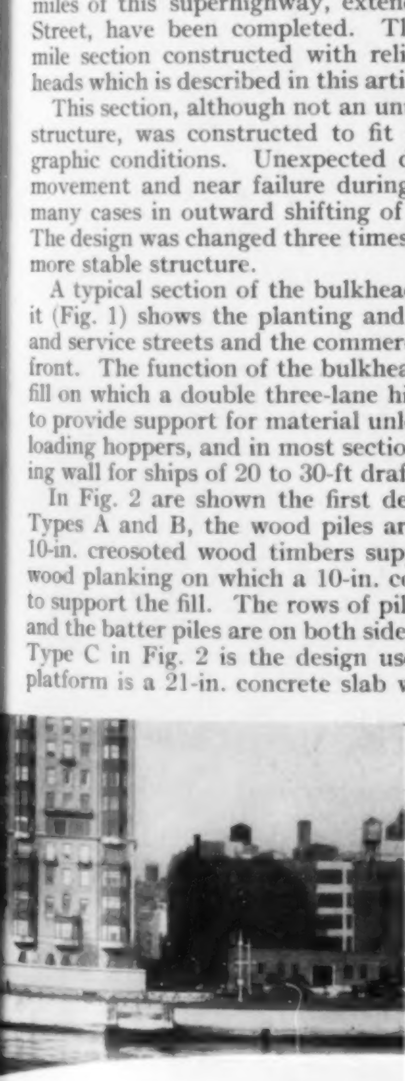
A LARGE part of the East River Drive was constructed on newly made land 100 to 250 ft wide. In the commercial areas an 80-ft strip next to the bulkhead line is reserved for material-handling hoppers and other industrial uses. The adjacent 70 or 80 ft is occupied by the six lanes of the Drive proper, and the remaining inshore area is developed as park area or sold back to the adjacent upland owners. Economy pointed to the use of a relieving platform bulkhead on vertical and batter piles, stabilized by riprap deposited outside the inshore edge of the platform. Of the several types tried, a final design was evolved using a heavy concrete platform located just above low tide supporting a bulkhead wall about 6 ft high to hold back the fill and act as a wharf. A sheet-pile cutoff wall simply prevents the fill from washing out under the platform, and the tie-rods connecting the platform to concrete anchors were frequently found necessary to assure the stability of the bulkhead. The $3^1/_2$ miles, as built, cost about ten million dollars. Many problems of design and construction are covered in this article, originally presented before the Metropolitan Section at its December 1941 meeting.

ONE of New York's most recent traffic improvements is the East River Drive and one of the important features of the project is an improved relieving platform bulkhead. Since 1938, five and a half miles of this superhighway, extending south from 93d Street, have been completed. This includes the $3^1/_2$-mile section constructed with relieving platform bulkheads which is described in this article.

This section, although not an unusual or novel type of structure, was constructed to fit rather unusual topographic conditions. Unexpected difficulties resulted in movement and near failure during construction and in many cases in outward shifting of the completed work. The design was changed three times in order to produce a more stable structure.

A typical section of the bulkhead with the fill behind it (Fig. 1) shows the planting and paving of the Drive and service streets and the commercial areas on the river front. The function of the bulkhead is to hold back the fill on which a double three-lane highway may be built, to provide support for material unloaded from ships and loading hoppers, and in most sections to provide a docking wall for ships of 20 to 30-ft draft.

In Fig. 2 are shown the first designs developed. In Types A and B, the wood piles are capped with 10 by 10-in. creosoted wood timbers supporting 2-in. or 4-in. wood planking on which a 10-in. concrete slab is placed to support the fill. The rows of piles are 4 ft on centers and the batter piles are on both sides of each row of piles. Type C in Fig. 2 is the design used up to 1940. The platform is a 21-in. concrete slab with the piles bedded 4 in. into the concrete and with no wooden platform. This design did not contemplate the use of any ties from the bulkhead to firm land, but when movement occurred, ties were constructed where found necessary.



Bulkhead Construction—Screen Wall and Boat Basin at River House, South of 53d Street

The ultimate design, a refinement of Type C, is designated Type D and is shown in Fig. 1. It embodies changes found advisable from experience with the original design. The thickness of the concrete platform is increased from 21 to 28 in. This allows a 12-in. projection of the piles into the concrete and also makes possible a more positive tie between the batter piles and the vertical piles. The piles are spaced closer in each row but the rows are 5 ft 6 in. apart instead of 5 ft, and the three batter piles are all on one side of each row. The resulting design provides for all the riprap that can be placed without filling the river channel above El. 30 and all the batter piles that can be effectively used, at the same time leaving more room to deposit the riprap.

In every section of bulkhead between expansion joints, one complete tie and anchor and one tie stub are provided as a standard part of the design. The single tie is usually adequate. The stub can be extended to a second anchor when movement is found.

There was a reason why this bulkhead had to be built differently from the usual type. If it had been in an undeveloped location where land values were low and where the position of the wall was not controlled by a previously established Government bulkhead line, the wall would probably have been placed as close to the shore as a navigable channel could be maintained. The platform would have been designed wide enough to reach to the existing shoreline and then the structure would not have had to resist any lateral pressure tending to move it out into the channel.

Such conditions, however, do not obtain at this location. The East River Drive bulkheads are constructed on the "bulkhead line" which has been established by the U.S. Army Engineers. This line is usually 100 to 250 ft offshore, in water 20 to 30 ft deep. The underlying material is quite uniformly 10 to 20 ft of soft mud over 10 to 20 ft of material that involves hard driving either to rock or to refusal in sand.

With this topography, a platform extending all the way back to shore would cost many times as much as the design adopted. A solid masonry wall along the bulkhead line would be unreasonably expensive because it would be 30 to 50 ft high, mostly under water, and supported on piles. A platform above the water level supported on either wood or concrete piles would be more expensive and less permanent. A relieving platform bulkhead is, therefore, the correct solution of the problem, even though the design must be somewhat indeterminate and frequently movement of the structure occurs.

The major problem was to design a structure which would withstand the lateral pressure of the fill and hold

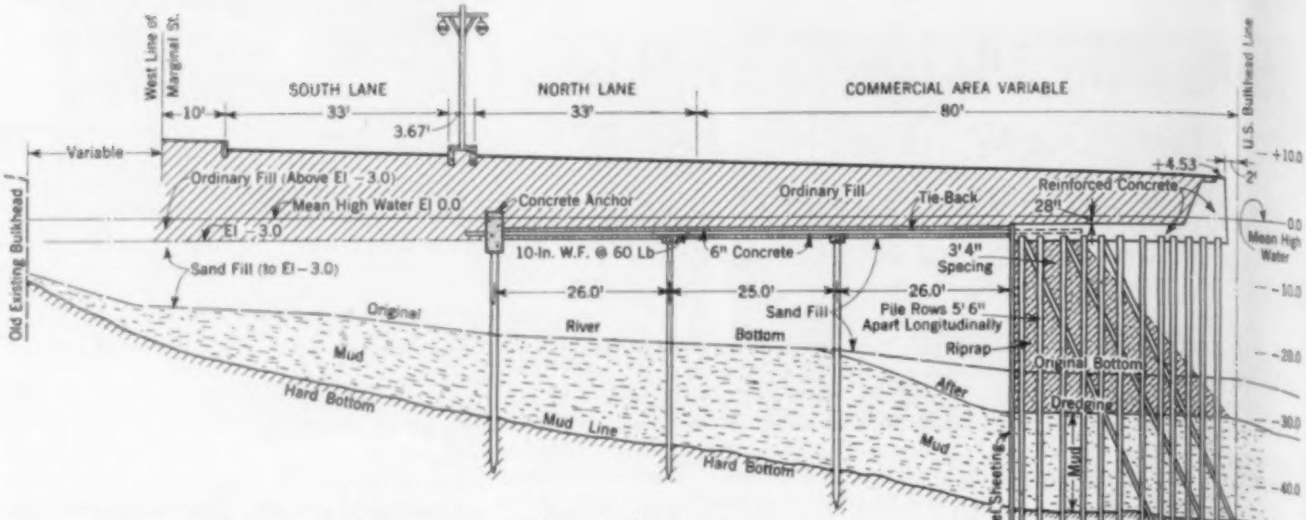


FIG. 1. TYPICAL SECTION OF WATERFRONT HIGHWAY SHOWING TYPE D RELIEVING PLATFORM BULKHEAD CONSTRUCTION

it from moving out, not only after the structure was completed and the fill compacted, but also during construction. The wall cannot be allowed to move after it is completed because important structures frequently must be built on the platform or in the fill back of the sheet piling, as, for instance, two viaducts and two sanitation dumps. At such places, it is important that all movement be overcome before the foundations for these structures are built. In other less vital locations, it is advisable to have the movement under control before any surface structures are placed.

Up to 1939, when the Type C bulkheads were built, using anchor and ties only when necessary, it was found that when 30 ft of water and a mud condition prevailed (Fig. 1), a slight movement, perhaps $^1/_8$ in. or more in two weeks, continued sometimes as long as one year after completion. It was impossible to determine in advance when such movement would occur. Probably it was caused by the loose condition of the riprap, because if the mass of riprap were solid and on a solid base it could readily withstand the sand pressure on the land side.

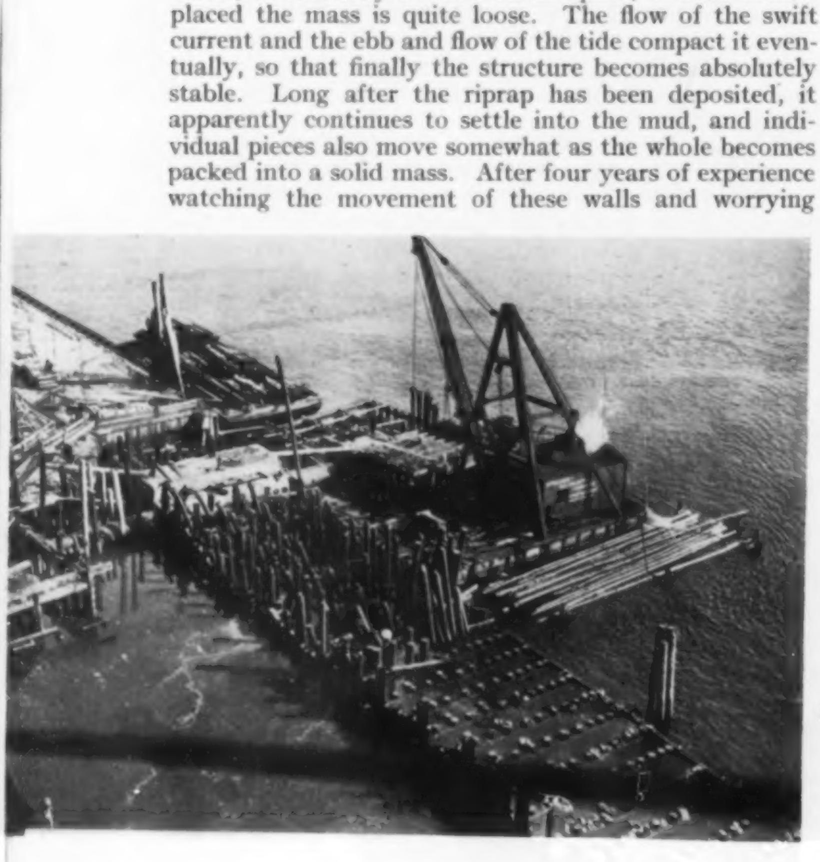
This riprap is deposited by dropping it at the water level from an orange-peel bucket. It sinks through the water, retarded by friction on the piles, and when first placed the mass is quite loose. The flow of the swift current and the ebb and flow of the tide compact it eventually, so that finally the structure becomes absolutely stable. Long after the riprap has been deposited, it apparently continues to settle into the mud, and individual pieces also move somewhat as the whole becomes packed into a solid mass. After four years of experience watching the movement of these walls and worrying about how to stop it, we developed the design now being used (Type D, Fig. 1).

This design depends on ties to supplement the riprap and batter piles, which by themselves frequently proved inadequate to prevent outward movement. Equally good results could probably be attained if the platform were made wider or if the fill back of the sheet piling were made of riprap, but the expense would then be much greater than ties and anchors, which cost only about $1,000 each. The reason widening of the platform results in greater cost than the proportional increase in the quantities is that after the platform becomes more than 36 ft wide, longer booms are required on the derrick boats to reach the work from the river side of the wall, and this increases the unit cost of the work.

The first step in construction is to dredge the full area of the platform to El. −30 (Fig. 1). This removes some of the mud and insures a 25-ft depth of riprap. At the same time, it removes high areas of riprap which have usually been built up under old pier locations and are difficult to drive through. The vertical and batter wood piles and the sheet piling are then driven. When the piles have been stay-lathed in place, riprap is deposited in front of the sheet piling and sand in the back, simultaneously. It is left to the contractor to coordinate the placing of the sand and riprap so that the sheet piling remains in line.

When the riprapping is completed, the concrete platform and the retaining wall are constructed. Next, the fill is placed to its final level in the platform area. Then the ties, tunnels, and other structures are built in the area between the platform and the upland. Finally the fill is completed to its final grade over the whole area. This part of the work is all let to one contractor. When the subsurface structure and fill are completed, a paving contract is let, including paving, drainage, lighting, planting, and all other work necessary for the completion of the Drive.

With this construction procedure, a condition occurs when movement is unavoidable. This is when the piles have been driven, the riprap placed, and the sand fill raised to a level where construction of the ties and anchors can start. At this stage practically all resistance



BULKHEAD PILES BEFORE AND AFTER CUTOFF AND STAY-LATHING

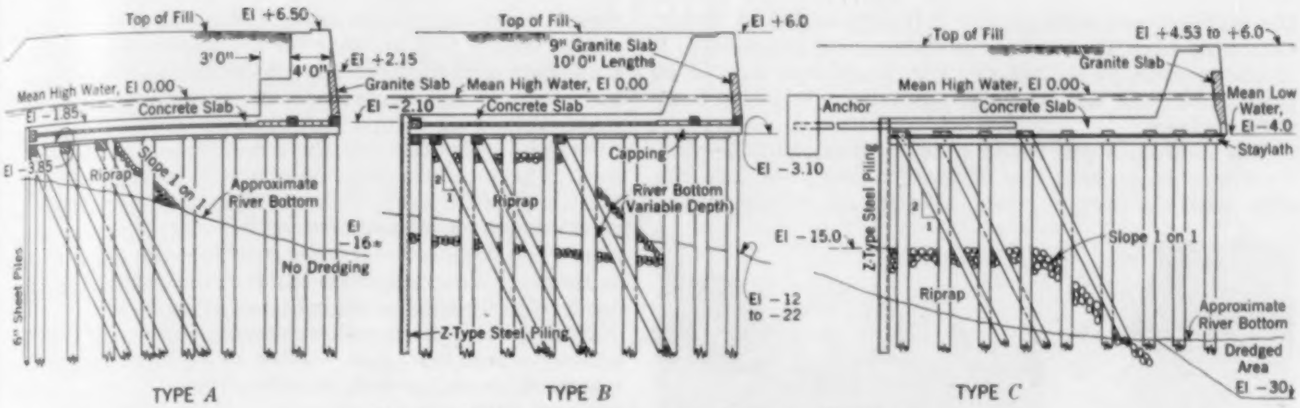


Fig. 2. Evolution of Relieving Platform Design

Type A, with 6-In. Wood Sheeting, No Anchors; Type B, with Steel Sheeting, Wood and Concrete Platform, No Anchors; Type C, Steel Sheet Piling, All Concrete Platform, No Anchors; Type D, Final Design (See Fig. 1), All-Concrete Platform with Tie-Rods for Auxiliary Support

to outward movement depends on the newly placed riprap. The batter piles furnish little stability because there is no vertical load on their tops. Hence outward movement always occurs, although it usually amounts to only 2 or 3 in. When the concrete platform and wall are completed and loaded with the fill, only a small movement continues.

In one case, before the platform was built, the whole system of piles, stay-lathing, and riprap moved out 2 ft in one night, 1 ft the next day, and still farther thereafter, up to a total distance of about 4 ft. In this case, the section which moved was confined to one area 80 ft long between two expansion joints. The cause was a combination of two unusual conditions. The piles there were very long, and the mud under the riprap was unusually deep; besides this, about 20 additional piles had been driven to support a tunnel. This close spacing of piles prevented the riprap from compacting enough to withstand the pressure of the sand against the sheet piling. When the sand fill was placed, the sheet piling, the riprap, and the wood piles were pushed out at the top.

This difficulty was remedied by constructing a special tie to the upland, which held the platform in its correct location. Then the two rows of piles that had moved beyond the platform area were abandoned and pulled out and the platform was extended in the rear of the sheet piling by means of additional batter and vertical piles to make up for those that had moved outside the bulkhead line.

In each section some movement has occurred after completion of the structure, but in no case has the base of the riprap slipped. The slope of the underlying rock is near enough level so that the resistance to sliding is prevented by the piles driven into 10 ft or more of sand. All this movement, therefore, has been in the nature of overturning—pivoting around the mud level, or motion allowed by the compacting of the riprap.

The maximum movement of the sections varied from $2^1/_2$ to 18 in., with an average of about 8 in. In several areas where ties were not used, the movement progressed for as long as one year after the bulkhead was completed. In all sections with ties it has slowed up and finally stopped after the wall has moved out a limited distance, the ties have become stressed, and the anchors have come to a solid bearing.

All the bulkhead designs are based on the assumption that creosoted wood piles will outlast the other elements of the structure, provided that they are cut off at mean low water and that the concrete is not exposed to frost and scouring. To obtain these conditions, the cut-off of the piles and the level of the exposed concrete are made as low as possible while still permitting the platform concrete to be placed in the dry in advance of the incoming tide. The elevation of the bottom of the concrete has therefore been established at about mean low water, or El. −4.0, although the tide goes below this level about 20 days out of every 30.

It has been found by experience that building of forms and placing of slab concrete can progress at the same speed as the driving of piles and other work which is a part of these contracts. Wall and platforms were constructed in sections not more than 80 ft long between expansion joints, and have a volume of concrete of 230 cu yd, which is more than can be poured in advance of the tide. Usually the contractor elected to make one construction joint in the middle of each section in order to meet the specification that all this slab concrete must be poured in the dry.

At their ends sections are held together by keys separated by 1-in. pre-cast fiber board. The keys allow slight relative vertical movement for possible unequal settlement of the piles, and prevent one section from moving out into the river beyond the adjacent section. In the earlier designs these keys were only 5 in. deep, but in the improved design (Type D) a larger key, 12 in. deep, was used.

The platforms are designed for the dead load plus live loads of 300 lb per sq ft in the area of drives and parks, and 1,000 lb in commercial areas. The vertical piles are spaced to carry 15 tons each. Locating the concrete wall 2 ft back of the established U.S. bulkhead line allows 12 in. for a fender system when required, and 12 in. for possible movement of the bulkhead. Within the tide range

Platform Forms Ready for Placing Reinforcing

the wall is faced with granite 5 ft high and 8 in. thick, extending from the bottom of the exposed concrete to 2 ft above mean high water. The stone was erected in place before the concrete was poured and was tied back with anchors.

The steel sheet-pile cutoff wall at the back of the platform acts to prevent the fill from seeping out into the river with the outgoing tide. Settlement of the fill has

TYPICAL BULKHEAD AND 53D STREET EXIT LANE

occurred at places where any opening was left. Driving the sheet piling about 5 ft below the penetration due to its own weight gives a sufficient toe hold.

Sand fill is specified up to low-water level. The remaining 6 or 8 ft above the relieving platform (Fig. 1) is ordinary fill from city excavations. As experience showed that there was not sufficient ordinary fill available, it was necessary to bring in extra sand from outside sources. A further reason was that usually sand is deposited hydraulically, thus causing less pressure on the sheet piling and less danger of disturbing the piles in the rear of the bulkhead, which are usually driven by floating rigs to support the ties, sewers, and other structures. If this fill were to be deposited by truck, it would be necessary to drive these piles afterward, with some delay and greater cost; otherwise they would be pushed out of line. All components of the ordinary fill are required to be less than 18 in. in size and so graded that they will make a solid mass.

The commercial areas of the Drive are paved with stabilized cinders. In the planting areas usually old granite blocks with seeded joints are used. Roadways are 33 ft wide with a steel-faced concrete curb, and are paved with 6-in. concrete surfaced with 3 in. of standard sheet asphalt. A 10 by 10-in. creosoted timber is bedded in the base along the outer edge of the concrete river wall. This assures more perfect concrete at this most vulnerable part of the concrete structure.

Ties are 10-in. 58-lb structural-steel H-beams encased in concrete with an anchor 80 ft back in the fill—a concrete block 12 ft long, 6 ft high, and 3 ft thick. These ties are not pre-stressed; the wall is allowed to move the necessary distance to stress them. Wherever there is a sewer or tunnel, which happens quite frequently along the East River Drive, its bottom slab is thickened and reinforced to act as a tie instead of the usual one. The ties are supported about every 25 ft on piles to prevent sagging caused by the settlement of the fill.

In commercial areas, the 80 ft adjacent to the bulkhead is reserved by the Dock Department for rental to various industries—60 ft for plant or buildings and 20 ft for commercial traffic. Inshore from this commercial area is a double three-lane highway, each lane 33 ft wide with a center mall 3 ft 8 in. wide on which the street lighting standards are constructed. Settlement of the paving and curbs always occurs because even though the fill is reasonably well compacted, it rests at the bottom on river mud. To take up the irregularities of the settlement, the 6-in. concrete base of the asphalt paving is reinforced with $^{3}/_{8}$-in. rods on 12-in. centers in both directions. Such a pavement settles fairly uniformly except when it passes over some solid structure like a sewer or tunnel. When uneven settlement occurs, the low spots are leveled up with an added layer of asphalt.

Piles are Southern yellow pine or Douglas fir with 14-in. butts and 6-in. tips. After being peeled, they are seasoned by steam and pressure treated in an air-tight cylinder with coal-tar creosote. A final retention of 16 lb per cu ft by the full cell process is required for the yellow pine, and 14 lb for the Douglas fir.

Where the overburden of the rock is insufficient to hold down the piles, a bed of riprap is deposited from bottom-dump scows to a depth of about 8 ft. It was found advisable to use riprap of a size that would pass through a $2^{1}/_{2}$-in. ring. Smaller riprap was likely to float away in the swift current, and larger sizes would probably damage the piles that had to be driven through it.

Steel sheet piling is of the Z-type, weighs 32 lb per sq ft, and has a $^{1}/_{2}$-in. thickness of steel at the flanges and $^{3}/_{8}$ in. in the webs. The top of the sheet piling is cut off level with the top of the platform slab and each individual pile is anchored to the concrete by means of an angle. Riprap is specified hard, uniformly graded, durable stone not larger than 18 in. or smaller than $2^{1}/_{2}$ in.

With this design of the bulkhead wall, the only concrete surfaces exposed to frost, salt water, or scouring action are the top of the wall and the bottom of the platform slab, which is at approximately mean low water. The creosoted wood piles supporting the structure are exposed only below the water level.

Before the Drive was built, some sections of this river front, even areas adjacent to well-developed residence districts, furnished asylum to many human derelicts as well as to large numbers of rats. Any summer night two or more men—scavengers with push carts—congregated to eat and sleep in the blocks that were not thoroughly fenced off. This condition is completely remedied with the relieving platform wall design. Any open type of construction would not accomplish this purpose so well.

The fills back of the bulkheads and viaducts on this highway have added 70 acres to the area of Manhattan Island, an addition which has been conservatively estimated to be worth $16,000,000. These reclaimed areas, for the width of the 32-ft platforms, are constructed to carry the load of any ordinary commercial use, and inshore from the platform area, piles for the foundation of any kind of structure can be driven.

The average construction costs per linear foot, for the $3^{1}/_{2}$ miles of bulkhead, were as follows:

	Gross	Bulkhead, Fill, and Paving Only	Wood Piles Only
Bulkhead contracts.	$558	$484	$143
Paving contracts.	76	76	...
	$634	$560	$143

The planning, design, and inspection were done by the regular organization of the Office of the Borough President of Manhattan. The work has been under the direction of Stanley M. Isaacs, borough president; Walter D. Binger, M. Am. Soc. C.E., commissioner; and the writer as chief engineer.

The Present Status of Three-Dimensional Photoelasticity

By Raymond D. Mindlin, Assoc. M. Am. Soc. C.E.

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A detailed knowledge of the stress distribution in structural elements and machine parts is of great importance to designers in many branches of engineering. The mathematical methods of the theory of elasticity have been applied successfully to the solution of many stress problems involving relatively simple shapes, but innumerable cases arise for which the mathematical tools are inadequate by themselves. Recourse must then be had to experimental procedures, usually supplemented by information drawn from the theory of elasticity. One of the most useful of the experimental techniques has been the photoelastic method. This method is based upon the fact that, when transparent materials are stressed, their optical properties undergo changes which can be measured and related quantitatively to the state of stress.

Until recently the photoelastic method was limited to the analysis of two-dimensional states of stress; but, in the past few years, two new methods have been proposed, and to some extent used, for studying three-dimensional stress distributions. These are known as the "freezing method" and the "scattering method."

The Freezing Method

In 1850 Maxwell[1] performed an experiment which has been developed, lately, into a very promising three-dimensional method. Maxwell took a hollow cylinder of warm gelatin and rotated the inner surface of the cylinder, with respect to the outer surface, through a small angle about the axis of the cylinder. He allowed the gelatin to cool with the twisting couple still applied and observed that the resulting double refraction in the gelatin represented an elastic stress distribution. He was unable to explain the phenomenon and the matter was apparently ignored until 1935 when Solakian[2] heated a solid cylindrical bar of Marblette, applied a twisting couple about the axis of the bar, and allowed the material to cool with the couple still applied. He then cut out a slice perpendicular to the axis of the bar and examined the plate in a polariscope. However, the resulting fringe pattern did not correspond with the state of stress which is predicted by the St. Venant torsion theory.[3, 7, 10] The discrepancy seems to have been due to the large deformations involved.[15]

Oppel in 1936[4] made a similar experiment with a block of Trolon. He heated the block, pressed a metal sphere against it, and cooled the block under load. He observed that slices cut from the cooled block exhibited a fringe pattern corresponding to an elastic state of stress. The principles underlying this phenomenon have been better understood as a result of its interpretation, by Hetényi[3] and Kuske,[5] in the light of the diphase theory of hardening resins.

According to the diphase theory, the hardened resin used in photoelastic tests is composed of two constituents, an infusible three-dimensional skeleton embedded in a fusible matrix. The infusible part constitutes only a small portion of the total volume and is formed from the fusible part by the chemical action called polymerization.

At room temperature the fusible part has a high viscosity coefficient and carries most of the load applied to the model with almost imperceptible creep; but at elevated temperatures the viscosity coefficient decreases so that the fusible part flows and gradually transfers the load to the infusible skeleton. The deformation under constant load therefore approaches a limiting value which is determined by the percentage of polymerized material and its elastic properties. The higher the temperature, the higher will be the creep rate of the fusible part and the more rapidly will the limiting deformation be reached, as is indicated in Hetényi's experimental curves, Fig. 1. At a temperature of 115 C, the limiting deformation is reached almost instantaneously for Bakelite BT-61-893.

That the polymerized skeleton is linearly elastic is demonstrated by Hetényi's stress-strain data at 115 C shown in Fig. 2. The linearity of the optical effect is illustrated in Fig. 3.

The explanation of the freezing method is accordingly that, at the elevated temperature, the elastic, infusible skeleton alone resists the applied forces. When the model is cooled under load, the fusible part freezes around the deformed skeleton and essentially maintains the deformation when the load is removed. The resulting model is thus in a state of deformation corresponding to the elastic state of stress at the elevated temperature. Since the equilibrium between the fusible and infusible parts exists over regions of molecular proportions, this state of stress is essentially undisturbed by careful slicing of the model, as is illustrated in Fig. 4. This figure also shows one of the chief difficulties in the freezing method, namely, that the strain which must be applied to obtain high double refraction is so large as to change seriously the shape of the model. This is because, while Young's modulus is decreased in the ratio of about 640:1 at the elevated temperature, the stress-

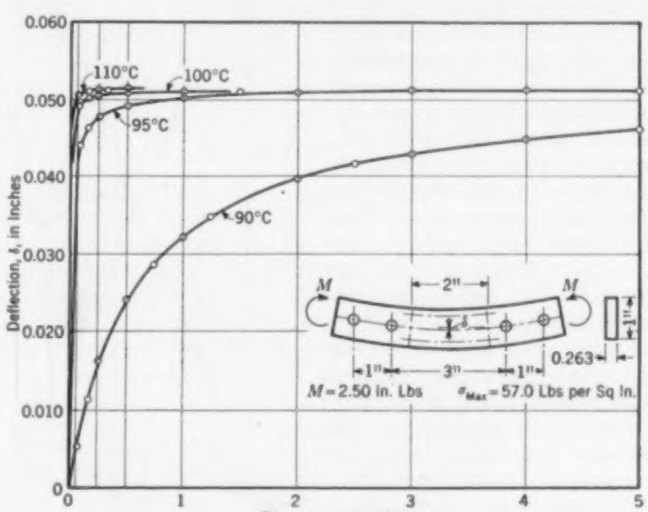

Fig. 1. Time-Deflection Curves for Bakelite at Elevated Temperatures (from M. Hetényi, Ref. 3)

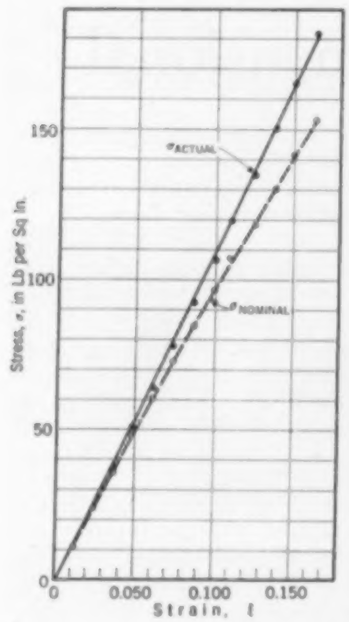

FIG. 2. STRESS-STRAIN CURVE FOR BAKELITE AT 115 C (M. HETÉNYI)

optical coefficient is increased only in the ratio of 26:1, for the most suitable material now known.

A typical study is shown in Fig. 5, by the freezing method, of a three-dimensional problem in stress concentration. The interpretation of the double refraction in the slice is rather more complicated than for two-dimensional stress systems. In general, the wave normal does not coincide with an axis of optical symmetry, so that the relative retardation observed is not a direct measure of a difference between principal indices of refraction, and is accordingly not proportional to a difference between principal stresses. As a similar difficulty is encountered in the scattering method, interpretation of optical observations will be discussed after the scattering method has been described.

THE SCATTERING METHOD

Weller[6] has described a three-dimensional method based upon the fact that, in a transparent medium, light scattered at right angles to the incident wave normal is plane polarized, the amplitude of the vibration reaching the eye of an observer being proportional to the projection of the light vector on the plane perpendicular to the direction of observation. The action at the point of scattering is similar to that produced by the analyzer in a plane polariscope (Fig. 6). If the light incident at a point is plane polarized, the intensity of scattered light observed at right angles to the wave normal will thus vary from zero, when looking along the direction of vibration, to a maximum, when looking perpendicularly to this direction.

In a stressed photoelastic material, light is transmitted in the form of rays whose transverse vibrations are directed along two mutually perpendicular axes lying in the plane perpendicular to the wave normal. These directions are called the axes of secondary principal stress.[7] The light scattered from any point (O, Fig. 6) may be considered as coming from two linearly polarized vibrations (along p and q) differing in phase by an amount depending upon the integrated optical properties of the material along the path from the point of entrance (O') into the model to the point at which the scattering

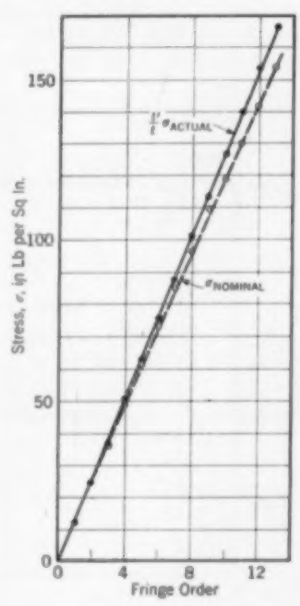

FIG. 3. STRESS–FRINGE ORDER CURVE FOR BAKELITE AT 115 C (M. HETÉNYI)

occurs. In general the two vibrations combine to form an elliptical vibration, the plane of the ellipse being perpendicular to the wave normal. The intensity of the light scattered at right angles to the wave normal will, in general, vary in intensity from point to point in the plane depending upon the projections of the ellipse of vibration associated with each point in the plane, the alternate light and dark regions forming a fringe pattern. Such an interference pattern will vary in distinctness and even in form as the direction of observation is changed, but in general it may be said that the distance between fringes in the direction of travel of the light is a function of the state of stress between the points of minimum intensity.

Weller's method of observation for a general state of stress is to fix attention on any point desired and rotate the model until the minimum distance between fringes straddling the point is obtained. If the stress field is perfectly uniform in the vicinity, this minimum distance gives the maximum principal stress difference at the point according to the simple relation[6]

$$\sigma_3 - \sigma_1 = \frac{C}{d} \quad \ldots\ldots\ldots\ldots (1)$$

where σ_3 and σ_1 are the algebraically largest and least principal stresses, respectively, C is the stress optical coefficient in stress units per unit thickness per fringe, and d is the distance between fringes as measured along the direction of the intermediate principal stress σ_2. The formula and reasoning are exactly the same as in two-dimensional work.

MATHEMATICAL INVESTIGATIONS

It is important to have available mathematical relations between stress and optical measurements which will either confirm or replace Eq. 1 for the general case of a non-homogeneous state of stress. Such relations can be obtained from studies of the solutions of the differential equations governing the propagation of light in non-homogeneous, non-isotropic media. However, solutions for only two special cases appear to be available and consequently the interpretation of the optical observations for the most general type of stress field is not completely understood at present.

The first non-homogeneous case to be solved[13] was that in which the orientations of the secondary principal stresses remain fixed, while their magnitudes vary linearly along the wave normal. This study showed that the stress computed from Eq. 1 is close enough, in this case, to the correct value for practical purposes. The conclusions from this study may be extrapolated to the case in which the secondary principal stresses, while remaining fixed in direction, vary in magnitude in any manner along the wave normal. Then the stress found from Eq. 1 will exist at some point intermediate between the two fringes and, for small stress gradients, the midpoint is close enough. If greater accuracy should be desired, plotting the fringe positions against their order will determine the point more closely.

The second special case investigated[8] was that in which the magnitudes of the secondary principal stresses remain constant while the angle between one of the secondary principal stress directions and a fixed direction perpendicular to the wave normal varies linearly with the coordinate measured along the wave normal. One of the conclusions found in this study was that Eq. 1 should be replaced by

$$\sigma_p - \sigma_q = \frac{C}{d}\left(1 + \frac{4\phi^2}{\Delta^2}\right)^{-1/2} = \frac{C}{dS} \quad \ldots\ldots (2)$$

for this case. In Eq. 2 σ_p and σ_q are the secondary principal stress magnitudes, ϕ/Δ is the ratio of rotation to retardation, and $S = \left(1 + \frac{4\phi^2}{\Delta^2}\right)^{1/2}$. It is possible that this relation may also be extrapolated to non-linear variations of orientation but this has not definitely been established.

No solution exists at present for a case in which there is a variation of both magnitude and orientation of secondary principal stresses. Until such a problem is solved there will exist some uncertainty as to the precise interpretation of the fringe pattern for the general state of stress in both the scattering and freezing methods. However, the interpretation is completely understood for planes of symmetry and for surface stresses so that useful applications are not prevented on this account.

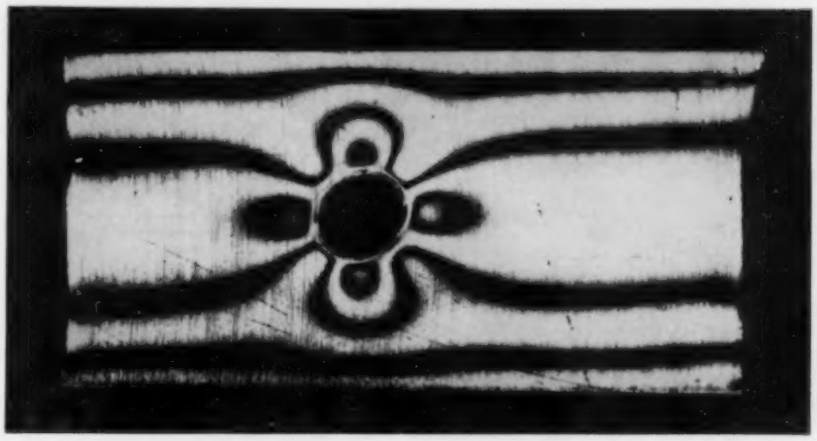

FIG. 5. LONGITUDINAL SLICE OF A BAR OF CIRCULAR CROSS SECTION WITH A TRANSVERSE HOLE, FROZEN IN TORSION (M. HETÉNYI)

One procedure for making the necessary optical measurements in the scattering method has already been mentioned. This is Weller's suggestion of universal rotation of the model, while under load, to determine the minimum fringe spacing at each point. There is an alternative procedure,[8] based upon the same principles as those used in the strain-rosette technique, in which all points in a plane section through the model are investigated simultaneously.

Whereas only three independent measurements need be made in the usual strain-rosette procedure, additional measurements may be required in a three-dimensional stress system. The most general state of stress at a point is specified by six quantities, e.g., three normal stresses and three shearing stresses across the planes on which the normal stresses act. The reason only three measurements are required in the usual strain-rosette procedure is that the measurements are made on an unstressed surface so that one normal component and two shearing components of stress are known in advance to have zero values.

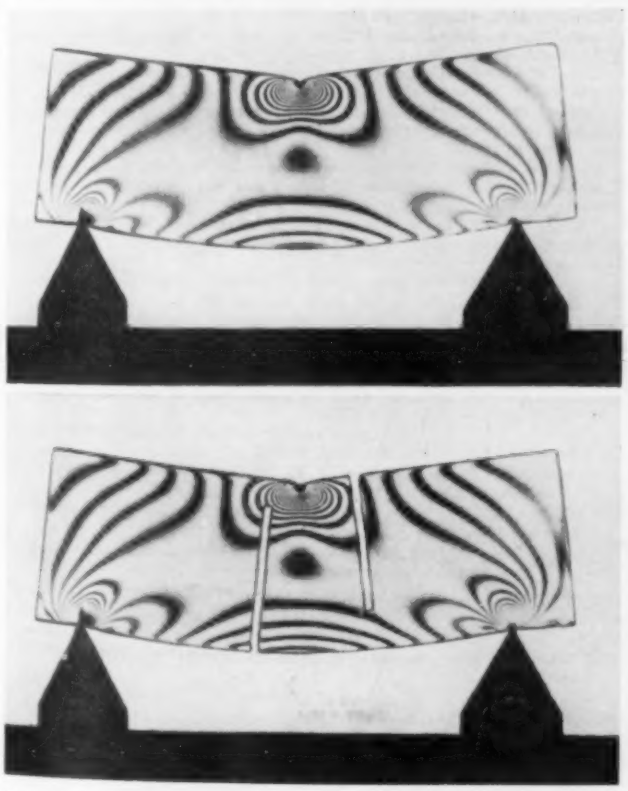

FIG. 4. SHOWING THAT CAREFUL SAWING DOES NOT DISTURB FRINGE PATTERN IN FREEZING METHOD (M. HETÉNYI)

As the effect of an isotropic stress is only to change the initial index of refraction, it cannot be detected by a photoelastic pattern and therefore only the three shearing stresses and the differences between the normal stresses, or the differences between the principal stresses and the planes on which they act, can be found. Without *a priori* knowledge as to the state of stress, five independent measurements must be made to determine the five unknown quantities.

If a plane sheet of circularly polarized light is passed through the model so that the wave normal takes three successive orientations in a single plane of the model, photographs of the light scattered at right angles to the wave normal for each of the three positions will show fringe spacings which supply three independent measurements for each point in the plane. Two additional positions of the wave normal in a perpendicular plane will give sufficient information to determine the five required quantities at all points along the line of intersection of the two planes. If, however, the orientations of the secondary principal stresses can be determined or are known, one need work only in a single plane, the three fringe spacings and the orientations being sufficient to determine all information for that plane. Such is the case, for example, when the plane under examination is a plane of symmetry, or when the state of stress is two-dimensional.

APPLICATION TO SPECIAL TYPES OF STRESS SYSTEMS

In plane stress, the magnitude and direction of a principal stress are known at every point. There are thus only three quantities to be determined, e.g., the two principal stresses in the plane and their orientation.

Fringe spacings observed at right angles to three orientations of the wave normal in the plane give all the necessary data. If the three orientations are taken 120° apart, the formulas for converting fringe spacings to magnitudes and directions of principal stresses become rather simple. They are identical with those developed for the equiangular strain-rosette.[9] Alternatively, two perpendicular measurements in the plane of the plate will give the sum of the principal stresses; and this information may be combined with data obtained from the usual two-dimensional fringe pattern to yield the individual magnitudes of the principal stresses.

A plane of symmetry or a principal plane in a three-dimensional case is very similar to the plane stress case. The quantities found from the three measurements at 120° will be the differences between the principal stresses and their orientations, which is all that one can expect to obtain by measurements of relative retardation.

THE FREEZING METHOD

The preceding development has been applied mostly to the determination of the state of stress from the patterns produced by the light scattered from a plane. It applies also with but few modifications to the "freezing" method. When the slice, obtained from this procedure, is placed in a polariscope, an integrated effect through the thickness is obtained. If the stress in the neighborhood of a point is uniform, the fringe order appearing there when the slice is in a circular polariscope is $n = \frac{(\sigma_p - \sigma_q)t}{C}$, where, as before, σ_p and σ_q are the secondary principal stresses in the plane perpendicular to the wave normal (which need not be the plane of the slice), t is the thickness of the slice in the direction of the wave normal, and C is the stress-optical coefficient. The difference between the secondary principal stresses is therefore determined directly from the fringe order, while their orientation can be obtained in the same way one obtains the isoclinics for a model under plane stress. If the fringe orders and the orientations are obtained for three different directions of the wave normal, all information possible as to the state of stress is determined.

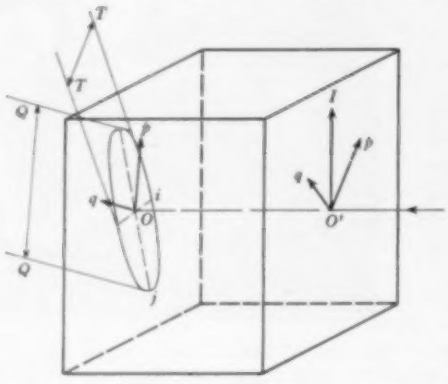

FIG. 6. DIAGRAM USED IN EXPLANATION OF SCATTERING METHOD

The method is entirely analogous to that described for the scattering method and applies with possibly even more accuracy because of the greater ease of determination of the directions of the secondary principal stresses. Little difficulty is caused by a small stress gradient unless the directions of σ_p and σ_q change along the wave normal, in which case additional measurements must be made[8] to determine the correction factor S.

FLAT SLAB ANALYSIS

Several methods have been proposed for the photoelastic investigation of bending stresses in flat slabs. The first of these to be applied successfully was that devised by Timby and Hedrick[11] and a second method is due to Goodier and Lee.[12] In both of these methods the two-dimensional optical technique is employed so that the methods are outside the scope of this review.

One method which has been proposed[13] for slab studies uses a three-dimensional technique based on the phenomenon that, when the rotation-retardation ratio (ϕ/Δ) is small, plane polarized light, entering the model so that the direction of polarization coincides with a principal stress direction, will remain plane polarized, with the direction of polarization coinciding with a secondary principal stress direction all the way through the model. An essential feature of the method is the "freezing" of a known initial stress in the slab before bending. The idea has been improved upon and successfully applied by D. C. Drucker.[14]

PRACTICAL CONSIDERATIONS

Both the freezing method and the scattering method for three-dimensional photoelasticity have been developed to a stage where they can be used for a wide variety of practical problems. If a two-dimensional apparatus is already available it will be less costly to convert it to an apparatus for the freezing method than to construct an apparatus for the scattering method. In fact the latter apparatus has not yet been standardized as there are at present two alternative techniques available, one involving universal rotation and the other using the rosette procedure, neither of which appears to have been actually applied to practical problems.

The freezing method and either one of the two techniques for the scattering method may be used in their present forms for analyzing stresses in planes of symmetry or on free boundaries. For general states of stress, further knowledge, both theoretical and experimental, is required for both the freezing and scattering methods before the quantities to be measured and their interpretation can be established definitely.

The most serious obstacle to the progress of three-dimensional photoelasticity is the lack of a suitable material for models. Because of this, the freezing method is in some cases open to objection because of the large deformations which occur at elevated temperatures with available materials, and both the freezing and scattering methods are troubled with initial stresses in the material.

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Trestle and Dam in Early Stage of Construction, Looking Downstream

Construction Plant and Methods—Friant Dam

By D. S. Walter

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On a large engineering project all parts of the work must dovetail into the construction—that gives the finished product. The reader who has followed the descriptions of aggregate and concrete production at Friant Dam in previous issues will be anxious to learn from Mr. Walter how all the various well-laid plans succeeded. His article covers the excavating procedure, the grouting, the trestle and concrete handling plant, the ingenious forms that eliminate interior bracing, and the placing in lifts of five 1-ft layers. This concludes the series of four papers on Friant Dam, continuous from the February issue.

Many difficult problems faced the Griffith Company and Bent Company, low bidder on the construction of Friant Dam, when the contract was awarded by the Bureau of Reclamation on October 9, 1939. The major difficulty was to secure adequate equipment for completing the work within a reasonable length of time. Although the specifications provided for 1,200 days, the need for speed was realized and construction equipment was purchased accordingly.

Several factors led the contractor to select a steel trestle with hammerhead and revolver type cranes in preference to cableways. The most important, and in all probability the deciding factor, was the topography at the dam site, where the distance between abutments at roadway level was approximately 3,430 ft. Although the contractor was in a position to take advantage of practical methods used on dams of similar construction, he had to work out many difficult problems for the first time, as Friant Dam was the proving ground for several construction innovations. Good judgment in plant design and selection of construction equipment has been responsible for the rapid progress attained to date.

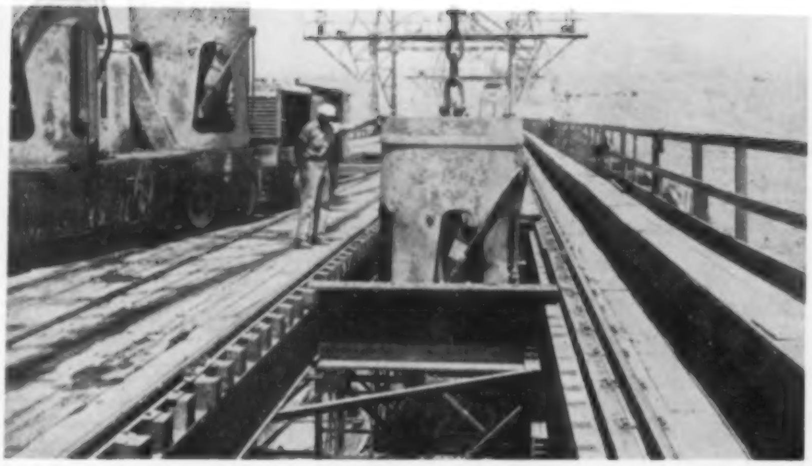
Preparing the Foundation

In general the rock exposed in the dam foundation was relatively hard and could be considered excellent in quality. The greater part of it was a quartz biotite schist which at many places had been impregnated with varying amounts of granitic material. The schistose planes strike at a low angle to the axis of the dam and dip steeply downstream. As the excavation progressed, an old fault was encountered to the left of the spillway section. The fault zone formed an oblique strip approximately 150 ft in width containing four gouge seams, roughly parallel. The rock between the seams appeared to be fairly hard but somewhat crushed and shattered. The excavation was carried deeper in this zone, and special treatment was given the seams. Several low-angle thrust faults and weather dikes encountered in the right abutment also required special treatment.

Trestle and Cranes for Concrete Placing, Looking Toward Right Abutment

For removing the topsoil and weathered rock, 14-cu yd self-loading carryall scrapers were used to good advantage after the material had been loosened with rooters and light blasting. As harder rock was reached, the excavation was completed with diesel-operated shovels of $1^1/_4$ to $2^1/_4$-cu yd capacity, loading into 8-cu yd trucks. One electric shovel of 5-cu yd capacity was used for several months. All excavated material was wasted upstream from the dam below low-water level, or if of good quality was stockpiled downsteam from the dam for use in future backfill.

Wagon drills, supplemented with jackhammers, were used for drilling the rock prior to blasting. The holes ranged in depth from 20 ft, spaced on 6-ft centers, where considerable overburden was present, to 6 ft or less,



Removable Hatchways Permit Concrete Bucket to Be Lowered Through Trestle Deck

spaced on 3-ft centers, near the final foundation lines. For loading the holes, 40% explosive was used. Air for drilling operations was supplied by a central compressor plant of seven units, with a combined capacity of 7,600 cu ft per min. Seven air receivers 4 ft in diameter and 12 ft long were located at various points adjacent to the excavation area, so that dry air substantially at full pressure (100 lb per sq in.) was available at all times.

Excavation, including that for the spillway apron, was carried down to sound ledge rock, free from weathered material, with a minimum of open seams and other objectionable defects. The necessary precautions were taken to preserve the rock below and beyond the lines of excavation in the soundest possible condition. As the excavation approached its final lines, the depth of holes and amount of explosive were reduced progressively. The firing of systems of blasts was controlled by the use of delay exploders.

Even when the rock appeared to be satisfactory, it was cleaned and carefully inspected to ascertain whether or not additional excavation was necessary. To properly key the dam into the foundation, excavation was carried somewhat deeper along the upstream and downstream limits so as to obtain sound rock for abutment of the fillets. At final bedrock level, all loose pieces were removed by barring and wedging, preparatory to low-pressure foundation grouting.

During the excavation of the dam foundation, a total of 49 exploratory holes $4\frac{1}{2}$ in. in diameter were drilled at various locations to assist in establishing final excavation lines. The cores obtained not only supplied important information during the period of excavation, but also supplemented that obtained in the original drilling and investigation of the dam site.

LOW-PRESSURE FOUNDATION GROUTING

For treating the foundation, low-pressure shallow grouting, followed by final high-pressure deep grouting, was provided. The shallow grouting consisted of three rows of holes, 25 to 30 ft deep, spaced on approximately 20-ft centers. The main cutoff or grout curtain consisted of deep holes drilled on approximately 5-ft spacing, on a line downstream from and parallel to the axis of the dam. At present the filling and grouting of these deeper holes is being performed from the gallery in the dam.

As the low-pressure grouting operations progressed, the regular pattern of shallow holes was extended to include all the old fault area exposed at the left abutment. Special holes were drilled to intercept the major seams in the foundation rock and other apparent zones of weakness. These holes were grouted after the holes in the regular pattern were completed. Numerous joints and fine fractures were present in the foundation rock, and every effort was made to fill these during the shallow low-pressure grouting operations in order to obtain a tight blanket for the deeper high-pressure grout curtain.

The grout holes, $1\frac{3}{8}$ in. in diameter, were drilled with pneumatically operated diamond-drill machines and grouted from centrally located stationary grout plants. Each plant consisted of a grout mixer, agitator, and pump, all pneumatically operated. Standard 1-in. pipe and 1-in. high-pressure hose were used. The holes were drilled and grouted in two or three stages, depending on the character of the surrounding rock.

It was customary to water-test each hole prior to grouting, at the same time calking the seams in the adjacent rock reasonably tight with lead wool, oakum, and wooden wedges. After the water test, grouting was started, using neat cement grout with water-cement ratio of 5.0 to 10.0, depending on the results of the water test. If any hole accepted grout at a fair rate, the ratio was gradually reduced until the hole was accepting the maximum number of sacks of cement per hour, at the maximum allowable pumping pressure of 150 lb per sq in.

GROUT ACCEPTANCE OF FOUNDATION ROCK

Frequently grout leaks developed in the foundation rock. This necessitated an immediate reduction in the pumping pressure in the water-cement ratio of the injected grout. However, it was the rule at all times to make every effort to secure the maximum grout spread in the surrounding rock. The grout acceptance of the foundation rock was relatively small, considering the number of holes drilled. However, one hole at the right abutment took over 1,600 sacks of cement. In the process of grouting the shallow low-pressure holes in the entire dam foundation, a total of 21,568 sacks of cement was pumped into 725 holes, with an average acceptance of 0.97 sack per ft of drilled hole.

Two working levels or benches on the left (south) side of the river are used by the contractor for the major portion of his construction plant. On the lower level at El. 355, about 30 ft above the original river level, are located

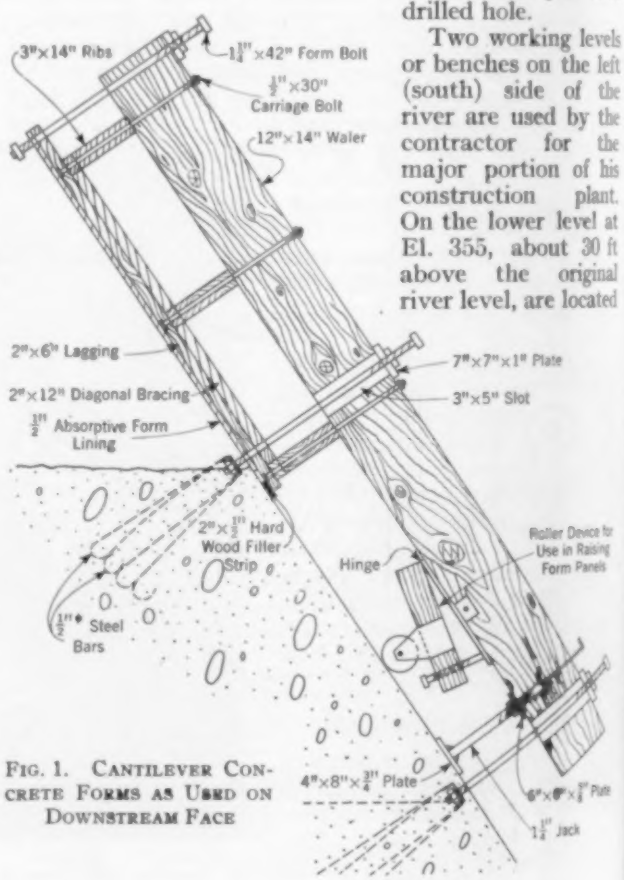


Fig. 1. Cantilever Concrete Forms as Used on Downstream Face

the machine shop, welding shop, warehouse, and miscellaneous buildings, in addition to the cement silos and cement unloading equipment. Standard-gage tracks on this level connect with the main line of the Southern Pacific Railway at Friant, one mile to the west. The upper level at El. 482 (the same as the concrete placing trestle) is reserved for the mixing plant, refrigeration plant, compressor plant, and miscellaneous shops and buildings. The carpenter shop, which was originally located on the 355-ft level, has been moved near the gravel plant some $2^1/_2$ miles downstream because of the requirement for additional space.

STEEL TRESTLE USED FOR PLACING MOST OF MASS CONCRETE IN DAM

Most of the mass concrete in the dam and appurtenant structures is being placed from the steel trestle located 129 ft downstream from the axis of the dam. The trestle is 2,430 ft in length and the deck level is at El. 482, which is 217 ft above the lowest bedrock foundation and 103 ft below the parapet of the dam. A timber approach connects the trestle with the mixing plant at the left abutment. The concrete-placing cranes operate on 110-lb rails of 38-ft gage, and two sets of tracks are provided for the concrete trains. Air, water, and power lines are installed near the upstream deck girder adjacent to an access walkway for the workmen.

The deck of the trestle is supported by tower sections located in alternate blocks of the dam, there being a distance of 70 ft between towers, except in the spillway section where the spans are somewhat greater because of the greater width of the blocks. The vertical columns comprising the tower sections consist of 14-in. wide-flange sections, ranging in weight from 95 lb to 158 lb, with considerable diagonal and horizontal bracing. For additional safety the higher towers were provided with 14-in. wide-flange diagonal wind braces or sway columns. The deck members, spaced on 38-ft centers, consist of 33-in. wide-flange sections through the towers, and 7-ft and 8-ft girders fabricated from plates and angles between towers. Cross beams consisting of 33-in. and 36-in. wide-flange sections are spaced at approximately 25-ft centers. Hatchways permit the buckets of concrete to be lowered through the trestle deck when concrete is being placed directly under the trestle.

Erection of the trestle was started at the left abutment and proceeded as foundation excavation permitted. For

SANDBLASTING HORIZONTAL CONSTRUCTION JOINT BEFORE PLACING NEW 5-FT LIFT

TYPICAL CANTILEVER PANEL FORM FOR DOWNSTREAM FACE ELIMINATES INTERIOR TIES AND BRACING

erection purposes one of the revolver cranes was used. Concrete footings for the tower columns and sway columns were placed on the bedrock as soon as the foundation was approved. Recesses formed around the columns and bracing members, where they project through the downstream face of the dam, will be filled with concrete when the upper portion of the trestle is removed at the completion of the contract.

The concrete-placing cranes on the trestle consist of two 294-ft hammerhead cranes and two revolver cranes equipped with $137^1/_2$-ft booms. These booms have recently been extended from their original length of 125 ft in order to reach the top of the dam. As the hammerhead cranes are extremely heavy, they are never operated closer than 40 ft to one another, thus preventing a dangerous concentration of load on the trestle. This minimum operating range is controlled automatically by means of electric eyes. Two stiffleg derricks with 180-ft booms are used for erecting and dismantling equipment and for placing concrete at isolated locations that cannot be reached with the cranes on the trestle.

For handling materials during the early stages of construction, a small trestle was erected 116 ft downstream from the main trestle. This service trestle, approximately 600 ft in length, carried a single track which connected with the railway through a spur track on the bench at El. 355. The purpose of the service trestle was to handle all materials and forms that would otherwise obstruct traffic and cause delays on the concrete-placing trestle.

Concrete from the mixing plant is discharged through a hopper directly into 4-cu yd bottom-dump buckets on cars. The cars are then moved out to the cranes on the placing trestle by small diesel-electric locomotives. Each locomotive handles one car carrying four full buckets, with space for an empty.

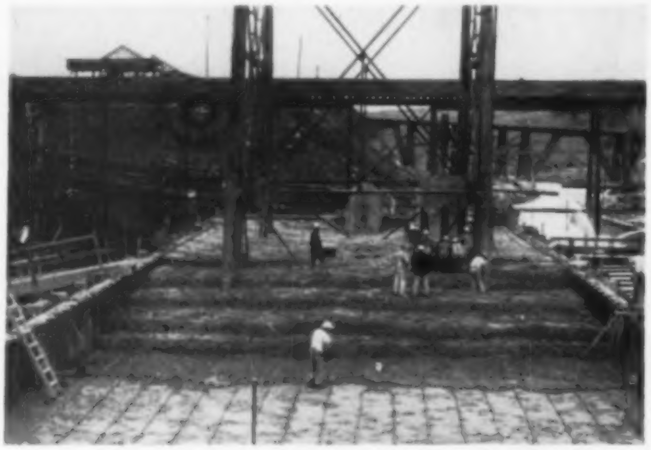
FORMS OF CANTILEVER TYPE

Friant Dam, a straight gravity-type structure containing approximately 2,150,000 cu yd of concrete, will have a maximum height of 320 ft above the lowest point at bedrock. Transverse contraction joints divide the dam into blocks 50 ft wide in the abutment sections and 56 ft wide through the center or spillway section. As longitudinal joints are not provided, the individual blocks vary in length from about 30 ft at the extreme abutments to a maximum of 270 ft in the central section.

The forms used for the upstream and downstream faces and for the contraction joints of the dam are of the cantilever type requiring no tie-rods or interior bracing for support and alinement. They are well adapted to the use of absorptive form lining, which is required on all sur-

faces that will be permanently exposed to view. The forms (Fig. 1) are held rigidly in place by means of 12 by 14-in. vertical cantilever walers, on $8^1/_2$-ft centers, extending down and anchored to the bottom of the previous lift. Accurate alinement is obtained by bolts and trench jacks.

For raising these forms (in 50-ft sections), four and sometimes five portable A-frames equipped with coffin-lever safety hoists are used. To prevent damage to the concrete surfaces when the forms are raised, heavy rollers are used under the cantilever supports. The minimum stripping time for the forms is 24 hours during the summer months, and 36 hours for the remainder of the year. All standard panel forms are prefabricated at the carpenter shop.



PLACING MASS CONCRETE—FIVE LAYERS EACH ABOUT 12 IN. THICK MAKE UP 5-FT LIFT

DETAILS OF MASS CONCRETE

Aggregate has a maximum size of 8-in. and each cubic yard of concrete contains 0.80 bbl of low-heat cement and 20% of pumicite by weight of the cement, except adjacent to the downstream face of the spillway, where 1 bbl of low-heat cement per cubic yard is used. The usual slump varies from $1^1/_4$ to $1^3/_4$ in., depending on the time of the year. Crews for placing the mass concrete consist of five men and a foreman. The concrete is placed in the blocks in horizontal layers 12 in. thick, there being five complete layers in each 5-ft. lift.

The maximum time interval between placing of the 12-in. layers is one hour; thus complete consolidation can be obtained between the successive layers while the concrete is still in a plastic condition. Each 4-cu yd batch is thoroughly vibrated with two large electric vibrators. In addition, the concrete adjacent to the forms is worked with a small electric vibrator to eliminate rock pockets or other defects missed by the larger vibrators. The minimum interval between successive 5-ft lifts is 72 hours.

At the completion of each 5-ft lift, the surface is left reasonably smooth with a uniform slope of approximately 6 in. from the edges of the blocks inward. The sumps at the center are drained by the risers of the wash-water drainage system, to facilitate the removal of cleanup wastes. During vibration the larger aggregates at the top of the lift are scattered out and "walked" into the concrete mass by workmen wearing "snowshoes" (pieces of plywood strapped to their shoes). This produces a dense surface, free of depressions and isolated pockets of rock, which can readily be cleaned by sandblast methods.

Prior to the placement of concrete in each block, the horizontal construction joint is wet-sandblasted to remove the loose or fractured concrete, and all laitance, coatings, and other foreign material. For this purpose stationary sandblast units, each consisting of a storage tank for dry sand and a pressure tank from which the sand is injected into the sandblast lines, are installed under alternate tower sections of the concrete-placing trestle. Each sand storage tank is located immediately below the trestle deck, its top flush with the floor of the trestle. The capacity of the tanks is approximately 8 cu yd, sufficient to clean from 4,000 to 6,000 sq ft of surface, depending upon the age and condition of the concrete. A spiral-welded pipe 6 in. in diameter carries the sand from the storage tank to its corresponding pressure tank. The pressure tank, 2 ft in diameter and 9 ft high, is maintained at a distance of from 5 to 25 ft above the concrete in the dam.

COMPRESSED AIR AND SAND FOR SANDBLASTING

Compressed air at a pressure of 100 lb per sq in. is supplied to each unit through a $1^1/_2$-in. air line equipped with a manifold for serving two $1^1/_2$-in. sandblast lines. Two 2-in.-diameter outlets at the bottom of the pressure tank, each with a suitable orifice, introduce sand through Y-connections into the sandblast lines. A $1^1/_2$-in. booster line connecting the main air line with the pressure tank causes the sand to flow at a uniform rate from the tank to the sandblast lines. The stream of air and sand travels through a $1^1/_2$-in. rubber hose to the special nozzle where water is added, producing the required mixture of sand, water, and air for the sandblasting operation. Three portable sandblast units are used at locations isolated from the stationary units.

Sand, as obtained from the gravel plant, carries more or less moisture, which must be removed before the sand can be used satisfactorily for sandblasting. For this purpose a rotary dryer with a capacity of 6 cu yd per hr is used. The dry sand is transported in trucks from the dryer to a larger hopper located at the left end of the placing trestle. A special flat car carrying 4-cu yd buckets transports the sand from the hopper to the storage tanks.

The average rate of sandblasting the horizontal construction joints is about 500 sq ft of surface per nozzle per hour. However, this time varies considerably with the season, age of concrete, kind of cement used, and condition of surface, that is, the degree of roughness and amount of slope to the sumps. During the sandblasting operation every effort is made to avoid cutting the surface too deeply, that is, to remove only the objectionable laitance and foreign deposits. The loose sand is then washed into the cleanup sumps by high-velocity jets of air and water. This material is effectively removed from the sumps through the wash-water drainage system.

For cooling, river water is pumped through pipes embedded in the concrete. The flow is started immediately prior to the placement of concrete around the pipes, so that the initial temperature rise in the concrete is held to a minimum. All concrete containing low-heat cement without pumicite is kept continuously moist for at least three weeks after placement. Where pumicite is used with the low-heat cement, the curing period is extended to four weeks. Automatic pipe sprinklers augmented by hand sprinkling are used for curing the concrete in the dam and appurtenant structures.

Friant Dam is one of the principal features of the Central Valley Project being built in California by the Bureau of Reclamation for river regulation, navigation improvement, flood control, salinity control, irrigation, and power production. It is scheduled to be substantially completed by July 1942.

Rejuvenating Wells with Chlorine

By Herbert L. White, M. Am. Soc. C.E.

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Difficulty was experienced in maintaining the flow of wells serving the University of Illinois. Rather rapidly the flow declined to about one-quarter of the original. Because of ground conditions and other evidence, calcium deposits were suspected, but treatment with muriatic acid disproved this theory. The next assumption was better—chlorine solution was poured in to attack possible iron growths, and the well was thoroughly flushed out. The resulting benefit, about 100% increase in flow, was convincing evidence not only of the presence of iron but also of the efficacy of chlorine for its removal. This paper describes the successful treatment of two wells in this manner.

By simple treatment it has proved feasible to renovate two sluggish water wells built for the University of Illinois at Urbana. These wells, with their gravel-packed screens, were constructed by a drilling contractor in 1935, and were a great success initially, but the specific capacity of one had decreased at the alarming rate of 75% in five years. Since the well and the necessary equipment, house, and discharge line cost about $25,000, the apparent loss in capacity caused considerable concern. The difficulty was increased by the fact that the static water level was dropping at the rate of about a foot a year. If both of these adverse phenomena had continued, the University would have been greatly handicapped in a short time.

Factors Complicating the Water Supply Problem

The situation regarding these wells, numbered 10 and 11 in the University system, was made more serious by the following factors:

1. The sand and gravel formation into which they were drilled was quite irregular and not suitable at many locations for the development of a satisfactory well.
2. Deeper water-bearing formations yield salt water.
3. Suitable surface water supplies are not available because the community is located at the headwaters of three streams.

Such a decrease in specific capacity (yield per foot of drawdown) is not unique with these two wells or in the University of Illinois well field, for other communities in Illinois, particularly in the central part, have had similar difficulties.

Workers Wear Gas Masks While Pouring Acid Into Steel Drum

To study the possibilities for a future water supply, a special committee was appointed by the University. It consisted of representatives of the Illinois Geological Survey, Illinois Water Survey, College of Engineering, Physical Plant Department, and the local water company. Many possibilities were considered but the only definite conclusions that could be drawn were:

1. The water-bearing formation is quite irregular and a few test holes should be drilled to find the most suitable location. (This work was postponed until an electric resistivity survey of the locality could be made.)
2. The wells in the north field of the local water company have yielded about twice as much water for the investment as have the University wells, in spite of the fact that the University wells are newer, have been operated at a lower rate, and are equipped with larger screens in an attempt to prevent incrustation of the screens.

Many factors are involved in the design, construction, and operation of wells which affect the investment, depreciation, and maintenance costs and thereby influence the ultimate cost of the water. Some of these factors are listed in Table I to enable the reader to visualize the problem better.

Previous Experience with Gravel-Packed Wells

Experience with earlier University gravel-packed wells will be related to indicate some of the problems encountered. Well No. 6, the first large gravel-packed screen well in the locality, was constructed in 1917, and

Table I. Factors of Design, Construction, and Operation of Wells Which Influence Cost of Water

A. Design details
- 1. Screen material
  - *a.* Non-ferrous metal
  - *b.* Ferrous metal
  - *c.* Concrete
- 2. Size of openings in screen
- 3. Total inlet area of screen
- 4. Proportion of screen constantly submerged
- 5. Composition of packing gravel
  - *a.* Limestone
  - *b.* Silica
- 6. Method of forming gravel pack
  - *a.* Natural (Johnson screen)
  - *b.* Artificial (Layne screen)
- 7. Casing weight and connections
- 8. Deviation of permanent well from test-hole log

B. Construction
- 1. Method of drilling
  - *a.* Rotary
  - *b.* Scow
  - *c.* Solid tools
- 2. Development of gravel pack
  - *a.* Overdevelopment
  - *b.* Underdevelopment
- 3. Method of placing screen
  - *a.* Bailing in
  - *b.* Pull back (drive casing to bottom of formation, set screen, and pull casing back)
- 4. Plug in bottom of screen
  - *a.* Screen driven into clay
  - *b.* Concrete plug
  - *c.* Wood plug
- 5. Alinement
  - *a.* Deviation from vertical line
  - *b.* Deviation from straight line
- 6. Leaks or weak places in casing

C. Operation
- 1. Excessive drawdown may clog screen
  - *a* $CaCO_3$ deposited in gravel outside of screen and on screen
  - *b.* Fine sand drawn into gravel
  - *c.* Influence on iron growths
- 2. Proportion of time in operation may affect formation of iron growth on screen
- 3. Frequency and type of repairs
  - *a.* Acid treatment
  - *b.* Chlorine treatment
  - *c.* Agitation with surge block
  - *d.* Remove, clean, and replace screen and add packing gravel
- 4. Unusual conditions such as introduction of air into well to permit operation of continuous water-level recorder
- 5. Fluctuation of water-level elevation in the formation

had many faulty design features. It was abandoned in 1933 as a result of:

1. Caving, which partly shut off the screen from the water-bearing formation and threatened the safety of a nearby building.

2. Entrance of shallow water, and at one time of creek water, into the well.

Well No. 7 produced about 600 gal per min from 1924 to 1935. In the spring of 1935, the yield dropped rapidly when the well was kept in almost continuous operation. Treatment with acid during the summer of 1935 resulted in a small increase, but during the work a break was revealed in the casing or screen. In 1939, an attempt was made to remove the old screen but this was unsuccessful. The old screen was then perforated and a 16-in. screen was placed inside. The yield at the conclusion of the repairs was 260 gal per min. The total cost of the work was $4,014.41.

Starting in 1924, Well No. 8 produced about 600 gal per min, but in December 1934 it was abandoned because:

1. Breaks in the inner casing or screen permitted gravel to enter.

2. The well was about 16 in. out of line, probably because of a cave-in.

3. The outer casing leaked.

4. There was uncertainty concerning the ground conditions and the strength of the outer casing, particularly in view of the nearness of a sanitary sewer, and a large cave at the bottom of the outer casing.

Well No. 9 produced about 550 gal per min in 1931, but this dropped to about 100 gal per min in 1940. The decrease was probably due to clogging of the screen.

With these experiences as a background, we were ready to tackle the problems of Wells Nos. 10 and 11, which were becoming progressively worse. Well No. 10 is about 160 ft deep and has a shutter screen 26 in. in diameter and 55 ft in length. The specific capacity of the well in 1935 was about 100 gal per min but this had decreased until in 1940 it was 24.5 gal per min per ft of drawdown. When the well pump is not in operation, the well contains about 40 ft of water.

Since the water in the sand and gravel formation into which the wells are drilled contains calcium and magnesium bicarbonate; since cemented sand and gravel were found in material bailed from the water company's wells during repairs; and since screens which have been removed from those wells were partially obstructed by deposits of calcium and magnesium, it seemed probable that the reduction in the capacity of Well No. 10 was due to deposits of calcium carbonate on the screen or in the gravel adjacent to the screen. In order to correct this difficulty it was proposed to treat the well with acid and to surge it with a surge block. The acid treatment was intended to dissolve the calcium and magnesium carbonate on the screen, and agitation with the surge block would work any fine sand in the gravel pack into the well, where it could be removed with a bailer.

The pump was first removed from the well. Then a surge block, bolted to a 40-ft length of 8-in. flanged pipe and suspended by cable from a clam-shovel machine, was run into the well. The surge block had small pieces of wire rope projecting around its periphery, similar to bristles on a toothbrush. The wire rope was provided to clean out the slots in the screen. However, the rope was old elevator cable and its wire was too soft for this use. After surging for three hours, about 2 cu ft of sand and small gravel was bailed out.

After the sand and gravel had been removed, preparations were made for pouring acid into the well. A steel barrel was cut in two longitudinally to make a flat reservoir to receive the acid. This reservoir was connected with a 2-in. horizontal pipe which was connected in turn with a 2-in. tee directly over the well. The upper branch of the tee, which served as a vent, contained 4 ft of 2-in. pipe capped with an elbow, and the lower branch of the tee connected to a $1^1/_2$-in. black iron pipe extending to within about 8 ft of the bottom of the well. Over 400 gal of 20° Baume commercial muriatic acid was emptied into the steel barrel in three installments. The first installment was discharged about 8 ft above the bottom of the well, the second about 19 ft above the bottom, and the third about 29 ft above the bottom.

Three hours after the last acid was added, the $1^1/_2$-in. pipe was removed and the well was surged for $3^1/_2$ hours. On the following day the well was bailed for one hour, surged for three hours, and then alternately bailed and surged for four more hours. About a half yard of sand and gravel was removed from the well by means of the bailer. Water bailed from the well was chocolate brown in color. The pump was then replaced in the well and a test made. This indicated that the specific capacity had been increased from 24.5 to 38 gal per min per ft of drawdown. This increase in the specific capacity was not satisfactory, and the problem was studied further.

When the well was put back in operation after the acid treatment, it was observed that the water had an unusually high chlorine demand, and it occurred to me that the same type of problem might exist at the bottom of a well as in the filter bed of an iron-removal plant. The cost and time required to try the effect of chlorine on the well was quite small, so a chlorinator was installed to discharge highly chlorinated water through a $^1/_2$-in. galvanized iron pipe into the well near the bottom. After 24 hours the well pump was started and a test revealed that the specific capacity had been increased from 38 to 50.2 gal per min per ft of drawdown.

PROBLEM OF WELL NO. 11

The increase in specific capacity of Well No. 10 obtained by the use of chlorine indicated that most of the difficulty had been due to some organic growth and not to incrustation of the screen by calcium carbonate. Therefore chlorine treatment was tried in Well No. 11 first. Chlorine was applied at the rate of 30 lb per 24 hours until a total of about 34 lb had been added. The well was surged at intervals during this period by means of a deep-well turbine pump which was left in place in the well. When the water was pumped to waste at the completion of the chlorination, it was colored only slightly with iron and had a slight chlorine odor.

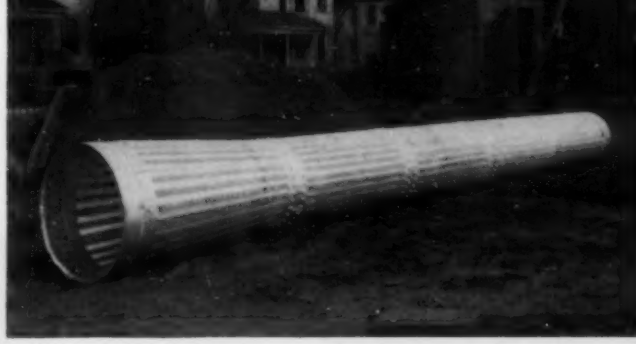
About 90 lb of additional chlorine was added and the well surged during this treatment. At the conclusion of the treatment the specific capacity had been increased from 34 to 60 gal per min per ft of drawdown.

The well was then treated with the same quantity of muriatic acid in the same manner as Well No. 10, except that the acid was added at only two elevations—5 ft and 16 ft above the bottom—because the screen is only 25 ft long. The acid treatment increased the specific capacity from 60 to 66 gal per min per ft of drawdown.

In November 1941 a test revealed that the specific capacity of Well No. 10 had dropped from 50.2 to 32 gal per min per ft of drawdown. After 80 lb of chlorine had been added to the well, the specific capacity was increased to 46.1 gal per min per ft of drawdown.

The interesting experience described in this article has convinced the writer that well screens or sand and gravel surrounding the screens may frequently be obstructed by iron growths. These growths may be readily removed, at very little expense, with solutions of chlorine.

Drilling Machine Sinking Well



Screen with Conical Section and Shoe

First Length of Casing, with Shoe

Riveting the Inner Casing

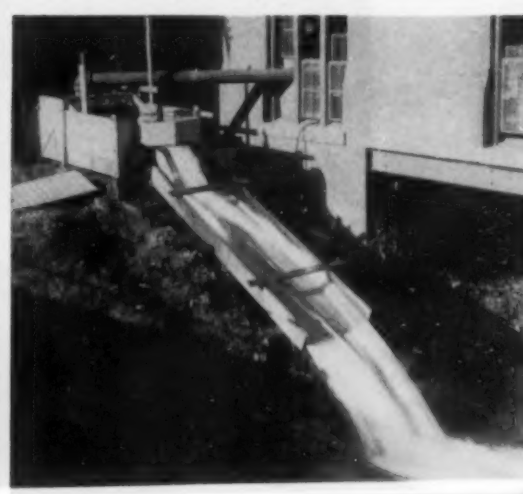
Welding Centering Guides to Inner Casing

Log of Construction Well No. 11

Test Pump Set-Up

Clam Shovel Rig Used to Set Pump in Well

Acceptance Test



Municipal Public Works Construction

Planning for Normal as Well as for Post-War Periods

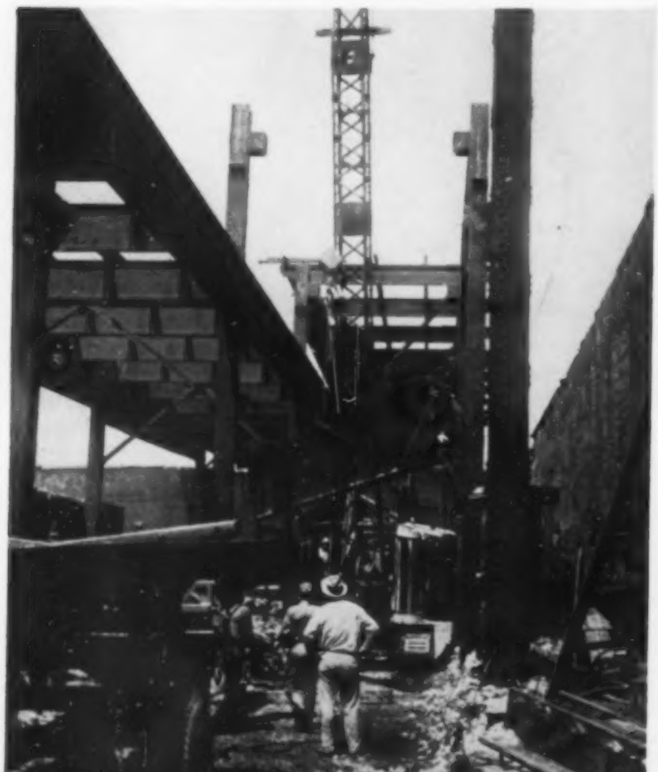
By A. C. Everham, M. Am. Soc. C.E.
Director of Public Works, Kansas City, Mo.

Keeping a modern city abreast of the times calls for unceasing vigilance. Attention to new developments and advanced civic ideals are only part of the task. Every advantage must also be taken of time-tested methods. All this calls for competent engineering assistance, supplemented by the best consulting service. Plans and programs for the post-war period are especially important. This paper, showing how well these problems are being analyzed and met in one large city, was originally presented before the session on "Preparedness for Post-War Conditions" at the 1942 Annual Meeting.

Many of the comforts of city life depend on public works. Such construction covers a large, interesting, and exceedingly important field. A review of the many services, conveniences, and accomplishments of a Public Works Department must bring feelings of justifiable pride to engineers trained in that work wherever such a subject is discussed.

When a new administration places an engineer in charge of a public works department, no matter how well he may have been trained in his previous experience, he will find much to engage his interest and test his resourcefulness. The ever-present changes and improvements in modern public works call for constant study. No sooner is one construction problem solved than new difficulties come up for consideration and planning. All engineers are conscious that construction methods are continually changing and advancing, and many can recall the time when they were very different from what they are today. Only about forty years ago reinforced concrete was unthought of; we built masonry structures. Pavements consisted largely of macadam and cedar blocks founded on planks and sand; sidewalks, of boards and asphalt; streets were lighted by gas lamps or electric arc lights; water for fire-fighting purposes was stored in cisterns; and countless other now outmoded methods have been replaced by new and better ones. Today we have so many new methods replacing the old that a public works official, to be efficient, must make continual investigations and studies to keep abreast of the times.

How the needs of a modern city call for changing types of construction is exemplified by the Michigan Avenue grade separation in Detroit. This viaduct was built in 1903 and 1904, using plate girders to carry the tracks of the Michigan Central Railroad over the street. When the project was being designed, there was considerable controversy and discussion, well publicized, because the plans called for supports in each curb line. Some public officials wanted the street completely spanned. This viaduct was completely rebuilt in 1941 with a support in the middle of the street. Who knows what form of construction may be used if this structure is rebuilt by another generation of engineers?



Obsolescence of Intercity Viaduct, Kansas City, Being Prevented by Reconstruction to Meet Changing Traffic Conditions

Municipal Work Must Be Long Lived

If there is one place where construction work must be well planned it is when it is done for a municipality. Where the work is important, adequate preliminary investigations should be made of the foundation, of the effect of weather conditions, of available materials of construction, and of records and experience obtained on previous work in the locality. In planning a new construction project, no engineer wants to build a permanent foundation under a building that he knows will be moved in ten or fifteen years. But when he knows that the building is to be of a permanent nature, it behooves him to use every possible means to insure its longevity.

A private corporation may be neglectful or build on a shoddy basis, but a municipality cannot afford to do so. The most unimportant part of the structure, most particularly where it meets the eye, must be constructed to stand up under the various deteriorating influences or the public is likely to lose faith in the structure and its builders. Handrails of concrete were used in the construction of many of the Kansas City Terminal Railway viaducts in connection with the Union Station development in 1911–1914. They were constructed in a manner that was not permanent. Since handrails are what the public sees, it is unfortunate that these have given the impression that the structures are not safe. Actually they are safe—they have stood up under the severest tests of loading, weather, and even smoke conditions on a very busy railroad.

It is indeed fortunate that the modern engineer has at his disposal a wide and varied engineering talent to

help him with his problems of building for permanency. If he fails to take advantage of the soils expert, the consulting engineer, and the testing laboratories, then he is shirking his duty to the public.

Perhaps it is because most of the best real estate has already been used for buildings that there is a growing demand for better borings, soil investigations, and soil loading tests. At any rate, the experts who have studied these problems have shown the fallacy of neglecting such investigations. Theirs is a worthy work; they are doing much that will prolong the useful life of structures.

EVER-INCREASING TRAFFIC PRESENTS A PROBLEM TO ALL PUBLIC WORKS DEPARTMENTS
Recently Completed Overpass on Southwest Trafficway Serves Congested Portion of Kansas City

Inadequate borings, or lack of borings, are also frequently the cause of lawsuits or claims which would never have been thought of if a suitable investigation had been made. Testing of materials, whether by contract or by city-owned laboratories, brings big dividends and helps to guarantee the sufficiency of various types of materials used on construction work. Private testing laboratories can be of inestimable value to a city.

In these times when adequate organizations are available to design structures after making soil analyses, borings, and tests, it is only good "horse sense" to use such organizations in connection with every important construction. With this type of service available at reasonable cost, it seems foolish for municipal authorities to start work without having made ample investigations.

NEED FOR CONSULTANTS AND TRAINED PERSONNEL

The employment of independent consulting engineers is essential for every important construction project. It is not enough for the taxpayers to depend solely on city employees, no matter how competent they may be, because their field is limited and there should be the most competent check on their services. Every public official should welcome the very best engineering advice he can get, not only for the benefit of the taxpayer, but also for his own protection. The consulting engineers of the United States can look with pride on their work for municipalities. Certainly no city is so well provided with experts that it can afford to ignore the excellent work of its consulting engineers.

Then, too, it is essential that an adequate personnel be assembled. Modern construction work is a highly developed, mechanized, and specialized process. It should be directed from top to bottom by employees who really know what they are doing. To get that type of employee in public service may not be easy. For best results at the least cost, engineers, inspectors, testers, and supervising forces for municipal construction work must be trained men—and that means technical training.

In 1940 Kansas City put into office a city manager, and he in turn has instructed the Director of Public Works to employ competent engineers, responsible only to the Director. The form of the organization makes it incumbent on each important commissioner to operate as an expert and to keep free from outside influences. If he or any subordinate does not work as he should, no political pressure can keep him in his position. Appeals to political bosses are of no avail—he either handles his work properly or his job is given to someone else.

This is something quite new in the city, and as a result construction work is being done better, more cheaply, and more satisfactorily to all concerned. Our construction procedures were hampered by many methods that had to be changed. For example, delivery tickets for materials were submitted for materials not furnished; short weights were common; manhole covers were stolen so frequently from outlying districts that where practical, reinforced concrete covers had to be substituted. Many other abuses too numerous to mention have been corrected. Suffice it to say that construction is now being handled by trained, competent men, all much interested in their work.

As soon as the type of construction has been determined, the design made, and a personnel organization developed, the next vital problem is to decide on the best method of handling the work. Shall it be done with city forces or by contract with an outside firm? For small jobs not requiring expert labor, or where time is an important factor, labor employed directly by the city is frequently and properly used. For work of magnitude, or where especially skilled labor is required, private contractors can give excellent service. In fact costs are apt to be lower on contract work than when city forces are utilized.

Many items of cost to a city do not appear in the record, but the costs are there just the same. While cities do not generally carry liability insurance on their labor, they do provide hospitalization, compensatory time allowances, sick leave, and vacations; and in other ways they pay indirectly a part of the insurance. Yet these items do not show in the city's cost estimate.

A contractor, if he wants to stay in business, must keep all these costs in mind when he makes an estimate. The competition among contractors is generally keen enough so that the average bid really gives the correct value of the projected work. Another reason for letting jobs by contract is that when work is slack, city construction employees paid on an annual basis may be less efficient, and some clever stalling on lesser projects can hardly be avoided. This of course is bad for the city and demoralizing to the men.

WPA LABOR SHOULD BE USED TO BEST ADVANTAGE

Still another method of constructing municipal utilities such as trafficways, street improvements, sewers, and other betterments is by the use of federal employees through such agencies as the Work Projects Administration. The amount of such help varies with the relief demand, and although it may become greatly reduced through the period of national emergency, there will always be, beyond any doubt, a certain number of men who are unemployable in private industry and defense

activities. The cost of this labor, while now assumed by the federal government, will of course be paid at some future time by the citizens of the entire country in the form of taxation. It therefore behooves each community to make a definite effort to secure a reasonable share of such labor and to employ it on well-planned projects which will be of lasting value to the community.

ENGINEERING FOR PERMANENCE DEMANDS STUDY OF FOUNDATION PROBLEMS
Construction View Shows Method of Providing Permanent Support for a City Structure

It is considered well worth while for each public works department to have in its organization someone well trained in the regulations and requirements of the federal government concerning the use of such labor and the supply of it that can be made available, as it can satisfactorily handle a great many types of municipal improvements. However, since the work is seldom carried out or supervised by the most competent employees, it is imperative that plans be prepared with great care and that strict supervision and guidance by public works officials and employees be maintained.

In this way pavements, sewers, and other simple projects can be very efficiently constructed at a cost to the community not greatly exceeding the value of the materials used. It should be stressed that the obligation rests entirely with the local public works executives to insure that this type of labor be employed on sound, worthwhile projects to avoid waste of federal funds on jobs of little or no value, created with the sole purpose of placing men on a relief payroll.

Of course, a well-planned program of construction, whether for normal or extraordinary times, is of no value without a financial plan to provide a method of paying for it. It is part of the public works official's job to find ways and means of obtaining the necessary money to finance the construction program.

In these days of stress and financial crises, it is difficult for private property to stand much more taxation for capital expenditures. So it is necessary to supply money by the sale of bonds or short-time securities unless assistance is obtained from the federal government. To administer work properly by any of these methods requires a consideration of the time element in the expenditures so that the money will be available as needed. This in turn calls for budgeting of all items in the expenditure of municipal funds.

A budget system can be adopted so as to handle not only this construction work but all work handled by a Department of Public Works. Without such a plan, money can be, and has been, improperly diverted to other uses. A modern budget system is simple to prepare, easy to follow, and provides monthly checks against the estimated expenditures.

When the war is over, countless men will return to our cities who will need jobs. Business will be changing from the manufacture of munitions of war to the manufacture of the goods for peacetime needs and comforts. During that reconstruction period, we will need many products not manufactured during war times. Men will be available for construction and reconstruction work. That will be a time to do work economically.

A RESERVOIR OF JOBS FOR THE POST-WAR PERIOD

The federal government and some cities are now preparing a vast reservoir of jobs for the post-war period. These jobs in municipal work consist of viaducts, bridges, repaving, hospitals, fire and water services, and schoolhouses. The government is planning to appropriate $50,000,000 as a revolving fund for engineering and other preliminary expenses in connection with this program. This money is to be repaid to the revolving fund from later appropriations when projects are finally financed and started. This should give work to many engineers not only after the war but in the immediate future.

Strikes are always settled and wars finally end. When this war is won, our job as engineers is to have designs ready and specifications written so that when our heroes come home from war they will find our cities ready for the improvements made necessary by the growing needs of the community and also for the structures that have been neglected during the pre-war period.

When the plans for post-war work are ready, the federal government, the municipal governments, and civil engineers will have done a real service to the people and to the heroes returning from the war—a service never before contemplated. Judging from history, the lack of such a plan has caused untold hardships to those who fought for their country in past conflicts.

We engineers and public works officials will not fail—we will have the plans ready. Our returning heroes and civilian defense workers must have complete assurance of jobs. What a fine accomplishment this will be! In fact, when present plans reach maturity, our plans for peace will be better than our present plans for war.

In Kansas City we have made a start on a six-year post-war program in the Department of Public Works. Other departments are also making plans. We have designs and specifications completed for one viaduct (pre-war cost estimated at $1,500,000) and three other projects of less importance, all in such shape that contracts for a total of $3,000,000 can be under construction in about thirty days after the money is made available.

The complete plan consists of 45 major street improvements, 8 bridges and viaducts, 23 sewer projects, 9 inspection stations for motor vehicles, the leveling of an unsightly block of ground adjacent to our Union Station, an armory, pedestrian subways, medial dividers on trafficways and boulevards, and a resurfacing and paving program alone estimated to cost $16,000,000. Surely from this program public works construction can be selected which will justify the federal government and the city in selecting enough work to keep many war veterans employed until industrial jobs become available.

We have been working on this plan for a year and we are convinced that at least some part of it will prove valuable in the post-war period.

Protection Aspects of Civilian Defense

By Sherwood B. Smith

Principal Engineer, Office of Chief of Engineers, Washington, D.C.

As a result of a recent trip to England I have gained a new insight into the nature and problems of aerial bombing. In particular my observations have a direct bearing on civilian protection.

Press reports have confirmed that there has been very little bombing in England since the spring of 1941, the Germans being occupied in Russia. Most of the damage done in the heavy bombing of the fall has been cleaned up, but little has been done in the way of replacing structures demolished by air attack. I was able therefore to get a fair idea of the extent and type of the damage. Although it is not permissible to go into details as to the particular damage seen, since it would be information of great value to Germany at this time, I can give general impressions without betraying the confidence of the British.

The most spectacular damage results from aerial mines. These are very large bombs in thin metal cases. They may be fitted with parachutes which permit them to float down slowly and explode above ground so as to produce the maximum blast effect. Another type has retarding vanes which serve the same general purpose. These mines are made as large as 6,000 lb in weight, but 4,000 lb is a more common size.

For demolition, high-explosive general-purpose bombs loaded with 40 to 60% explosive are used. These may be employed in conjunction with incendiary bombs to disrupt fire-fighting activities. In size they vary from 125 to 6,000 lb.

Incendiary bombs are of two general types. First, there is the electron bomb weighing one kilogram (2.2 lb), which can be dropped in large numbers and which burns with an intense flame after impact. These bombs sometimes have a small explosive charge which spreads the burning material so as to increase the chances of causing a conflagration. The liquid, scatter-type, incendiary bombs vary from about 30 to 600 lb in weight and contain a small explosive charge, an igniter, and a combustible liquid such as oil. On impact, burning oil is thrown over a considerable area. There are several other types of bombs but they are of no particular interest from the standpoint of civilian defense.

In London I saw areas in which all the buildings had been flattened by the terrific blast from one aerial mine. These structures were of the wall-bearing type and were not particularly well constructed. Generally speaking, the blast from a near miss causes the collapse of wall-bearing buildings, but has very little effect on steel or reinforced concrete frame structures. In the latter case walls between columns may be blown out, but usually the frame is not damaged.

Blast does a tremendous amount of damage in breaking glass. This is one of the major sources of casualties and injuries to occupants of buildings. Windows may be broken within a radius of 200 ft or more from the point of explosion.

Reporters of English civilian defense activities are unanimous in praise of the plucky, resourceful, and thorough job that has been done. This, as Mr. Smith confirms, applies not only to fighting the danger but to the mopping up and reconstruction that follows. He offers two new ideas—that aerial mines are becoming the greatest menace, and that the human system can withstand much greater blast pressures, up to 100 lb, than was formerly believed possible. This address was delivered before the December 9 meeting of the Philadelphia Section of the Society.

Neatly Sliced Out—High-Grade Residential Building Has Been Blasted by Bomb Exploding Inside

General-purpose bombs cause similar damage by blast but to a lesser extent than the aerial mines. Bombs exploding after penetrating the roofs or walls of buildings create tremendous havoc. In the case of wall-bearing buildings, complete demolition usually results. Damage to steel or reinforced concrete frame structures will be serious but localized. In addition to the effect of blast, the bombs cause considerable destruction by fragmentation.

Serious damage from general-purpose bombs comes from the effect of earth shock when the bomb explodes after it penetrates into the ground. This causes a wave of earth to move outward from the explosion with a very high rate of acceleration within distances up to one hundred feet for a large bomb; the result is lateral displacement of column and wall footings. In the case of a wall-bearing structure, the inward movement of the wall footing will cause the wall to fall outward and the structure to collapse. In the case of a steel or reinforced concrete frame structure, properly designed, the displacement of the column footings will probably not be serious.

Earth shock is a major factor in damage to underground utilities, especially gas mains and sewers. Power cables and telephone cables suffer less damage. Water mains are burst near the explosion and sometimes at remote points by an effect similar to water hammer.

As is well known, the greatest danger is from fire. In the more congested areas fires may be started by large-scale attacks with incendiary bombs. Before the fire departments can bring the fires under control, large areas may be destroyed. In England fires have frequently resulted from the congestion of firetrap buildings and difficulties in reaching incendiary bombs in time. Steel-frame buildings, while more resistant to the effects of explosive bombs, are more seriously damaged by fires unless structural members are adequately protected.

Casualties result from a variety of causes, a great number from secondary effects, that is falling debris, flying fragments, drowning, and

PROTECTION BUT NOT PREVENTION—FIREMEN DEFEND BUSINESS PROPERTY
Bomb Exploded Under Pavement Causing Collapse of Building at Left, Failure of Gas Main, and Ignition of Escaping Gas

suffocation. Of course persons close to the explosion are killed by blast. Those at a greater distance may escape this, but may be killed by flying splinters. It was interesting to learn that the blast pressure required to kill a human being has been found to be far in excess of that originally thought necessary. The human body can withstand instantaneous pressures up to 100 lb per sq in. Injury results from the force of the blow on the chest, causing hemorrhage of the lungs. Pressures entering the nose and mouth have relatively little effect.

GLASS CONSTITUTES A SERIOUS PROBLEM

What can be done for protection against the effects of bombs? Existing structures may be modified so as to give the best possible protection. The first thing to consider is flying glass. All glass from interior partitions should be removed and replaced by some other material which will not cause injury. Exterior window openings should be bricked up or closed with other splinterproof material, if sufficient light can be provided inside. Upper windows and skylights and lower floor windows which cannot be blocked up for other reasons should be treated so as to minimize the danger from flying glass. A most effective treatment for this purpose is the use of cloth pressed into varnish so that the whole glass and the inside of the frame is covered. In addition, wire netting should be used to catch larger pieces.

Glass constitutes a special problem in the case of factories because of the usual practice of using roof construction having considerable glazing. Obviously this constitutes a great danger from falling glass. Maintenance of blackout is difficult in cases where the glass is treated to make it opaque instead of using material which is reasonably resistant to blast, as any damage to the glazing destroys the blackout. If possible, all glass should be removed and replaced with a safe substitute; otherwise burlap held to the glass with emulsified asphalt, and wire netting, should be used to lessen the chances of glass falling on workers below.

Very little can be done to prevent the collapse of wall-bearing structures as a result of bombs detonating close by or in the structure. Buildings may be materially strengthened by bracing floor beams of lower floors so that they can withstand greater debris loads. In factories the machinery can be so arranged that all the units engaged in the same process are not located in one area; then the entire chain of operations will not be disrupted by one bomb hit. Damage can also be localized by the use of what are called dwarf walls. Such walls are just high enough to give lateral protection to men and machines and have the advantage of permitting overhead movement of material by crane.

Protection of utilities is primarily a question of minimizing interruptions to service by using link systems and by having the replacement units and material for repairs at strategic locations. Hydroelectric power plants and transmission lines are not easily damaged. The more vital units can be protected from splinters by sand bags or by splinterproof walls. Transformers and substations can be given similar protection. It is quite obvious, however, that complete protection of the system is out of the question. Therefore the emphasis must be placed on a well-organized scheme for making rapid repairs. This may require mobile units for replacement of those damaged by bombs.

Gas tanks are not so severely damaged as might be supposed. Even in cases of direct hits, tanks are not blown up, although the escaping gas may catch on fire. In most cases the tank can be put in operation after a reasonable period for repairs. No special protective measures, therefore, seem justified. The installation of cut-out valves and the arrangement of the piping so that a minimum area will suffer interruption to service, along with adequate organization for quick repairs, seem to be the best means of taking care of gas mains.

The same applies to water services, except that an adequate supply of water must be available in case of fire. Construction of a large number of static tanks to supplement the regular supply and to provide against possible damage to regular lines is a necessary step. Reservoirs and the other large units of the water supply system are not easily damaged and it is not believed that special protective measures are required, except at certain vulnerable points.

The European practice of making permanent repairs to all damaged water and gas mains and electrical

DESTRUCTION VIEWED BY DAYLIGHT, REMAINS OF A GROUP OF SMALL HOMES LEVELED BY AERIAL MINE

power mains before filling in craters and repairing streets has definite advantages; there is no assurance that the same section of piping will be damaged by the next bomb; and permanent repairs permit the street to be put back into full operation. Also this method costs less than making the repairs in two steps.

Bridges suffer very little on the whole. Damage is generally confined to the deck and minor structural elements. However, we have a large number of suspension bridges, which might require special treatment since the cables where they pass over the saddles and where they enter the anchorages are particularly vulnerable to a direct hit from a bomb.

There are very few serious interruptions to railway traffic. Most of the damage can be quickly repaired by filling in craters and replacing track and signal systems. Stations may be hit but can be used after the debris has been cleaned up and the track repaired.

The underground railways in London were for the most part constructed by the shield method, at much greater depths than those in the United States. They are therefore quite safe. We would have a much more serious problem because most of our subways were constructed by the cut-and-cover method and have limited overhead protection.

The Question of Fire Protection

Protection against fires is primarily a question of providing a satisfactory organization and the necessary equipment to deal with incendiary bombs promptly and to handle the large conflagrations which may occur if fires get out of hand. An adequate fire watch composed of those familiar with the residential area, or the manufacturing establishment, for which they are responsible is essential.

For putting out incendiary fires in their early stages the stirrup pump has been found to be a very satisfactory piece of equipment, but quick action is essential. In manufacturing establishments there should be a large amount of equipment of the portable type not dependent upon the general water supply, such as approved chemical extinguishers or sand. Static tanks should be provided in case the general water supply fails. Obviously adequate cover should be given to steel members, and other precautions recommended by underwriters for fire protection should be met.

In Europe it has not been found practical to provide protection against direct hits from bombs. The furnishing of shelters which would give complete protection would be tremendously expensive. It is doubtful if such a program could be accomplished during the period available without seriously interfering with the war effort. The Anderson-type shelter, intended for use in the back yard of a small house-owner, has proved very satisfactory from the standpoint of protection. It is of corrugated sheet iron with a very light earth cover. These shelters have not been satisfactory, however, from the standpoint of comfort for their occupants, who have been obliged to spend night after night in their cold and damp interiors. Shelters inside buildings, particularly in the basements of older houses, have not proved satisfactory because of the heavy debris loads, and because of the danger that those in such shelters will be trapped, drowned, or asphyxiated.

Surface shelters constructed in the streets are now considered to be the most satisfactory type of outside shelters. These are of reinforced brick with a reinforced concrete top—and in some cases, a reinforced concrete floor. They are designed to provide for 25 to 50 persons and are well tied together with reinforcing so that in case a bomb detonates in the earth nearby the whole structure will be moved without failure or injury to occupants. An asphaltic damp-proof course is inserted to destroy bond and permit the movement of the shelter with less friction.

The Morrison shelter provides a most satisfactory solution to the problem of providing reasonable protection and comfort. It is a rectangular box made of angle iron with a $^1/_8$-in. steel-plate top and expanded metal sides and ends. It is placed on the floor and is occupied by two persons sleeping on a mattress. This provides reasonable protection against debris, and since it is below the level of window sills it also gives some protection against splinters. These shelters can be used as a table during normal times and cost only about $100 each.

Tremendous Effort Required to Damage a City Seriously by Bombing

The most outstanding impression obtained in my recent visit to England was that the effort required to produce damage to a city by bombing is tremendous. When it is considered that the cities of the British Isles are within comparatively easy reach of the enemy, and that the enemy possessed a very large air force and was able to carry on sustained raids, it is surprising that the damage was not greater. I do not mean that certain sections of the city of London and other cities of the British Isles have not taken a considerable amount of punishment, but I do feel that looking at the whole picture, the damage has not been serious. It must be remembered that last fall the R.A.F. had not been built up to the strength of the Luftwaffe and that the organization for defense, both active and passive, was not so adequate as it is at present. It is the opinion of people with whom I have talked who have been in England, and in Germany, during the raids, that attacks on the civilian population are not justified by the results obtained.

The greatest danger from air attack is probably the possibility of hysteria on the part of the population. It is believed that our primary effort here in the United States should be to set up an adequate organization, improve fire-fighting facilities, and make the necessary engineering studies so that in the event of attack we will be able to take care of the situation calmly and effectively. If we prepare now and keep our heads, we should not suffer much from any attack that can be launched against the cities of the continental United States at present.

Collapse of Building Due to Earth Shock from Bomb Exploding Under Pavement near Front Wall

Note Earth on Pavement and Lack of Blast Damage to Windows

Engineers' Notebook

Ingenious Suggestions and Practical Data Useful in the Solution of a Variety of Engineering Problems

Irrigation Conduit Gunited on Simple Movable Forms

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A CONDUIT, 4 ft in diameter, built up by guniting a semicircular invert and then shooting the arch section over removable forms, has been constructed by the Irvine Company to convey irrigating water to the lands of its San Joaquin Rancho in Orange County, California. The method of construction, which was carried out at a rate of 330 ft of invert or arch in eight hours, has been found a most satisfactory solution of the problem of conduit construction.

After completion of the Santiago Dam in 1932, the Irvine Company and the Carpenter and Serrano Irrigation districts were faced with the problem of providing a conduit. The first part of this conveying system was a gunite-lined canal from the reservoir to the Peters Reservoir, where the Irvine Company regulated the daily flow to meet irrigation requirements on its valley lands. This canal is about 3 miles long, and was constructed under contract in 1933.

After one year of service, it was found necessary to cover about two miles of this canal because it passed through the county park, and visitors enjoyed throwing in rocks, sticks, and trash. Further, because of the sharp curves and the roughness of the surface, the desired capacity of 60 cu ft per sec was difficult to obtain. As a result the canal was covered with a flat arch section, which kept out the debris formerly thrown into the canal, and increased the capacity to 66 cu ft per sec.

To extend this conduit the company purchased a tractor-mounted compressor and gunite equipment. As use of pipe would be too expensive, it was decided to construct an experimental section of closed conduit, built in circular form. A 1,000-ft length was built to a 4-ft diameter, and the inside of the conduit had a finished appearance resembling that of good quality concrete pipe. After a year of service, the only cracks found were along the bond line between the invert and the arch cover. This difficulty was corrected in the later work.

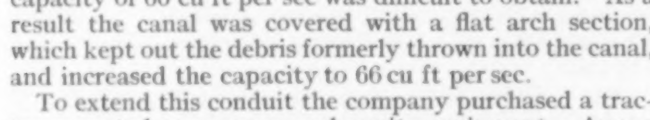
During the 1938 season, we constructed about 2¼ miles of this type of gunite conduit at a cost of $2.52 per lin ft, including excavation, trimming, pipe construction, and backfill. This cost is estimated to be approximately one-third the cost of a concrete pipe with equal capacity. Wages varied from laborers at $3.25 to nozzle men at $7.00 and foremen at $8.00. There were 11 men in the excavating gang and the same in the guniting crew.

Of course the line was built under the most favorable conditions—through alluvial fill, sandy loam soil, with very few sand lenses, and with easy access to materials and equipment. Backfill over the arch varies from 4 to 6 ft. It was considered impracticable to construct a pipe smaller than 48 in. in diameter, because the cost of removing forms would more than offset the saving in materials and guniting.

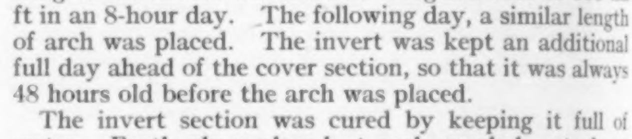
In general, the procedure was to open the trench with a "backhoe" bucket of special design (working on a 22-in. radius, leaving 3 in. for neat trim on both sides and bottom); to hand trim the semicircular invert section; to place the wire-mesh reinforcing; and then to "shoot" the 2 in. of gunite for this first half of the conduit. The guniting of the invert was done at a regular rate of 330 lin ft in an 8-hour day. The following day, a similar length of arch was placed. The invert was kept an additional full day ahead of the cover section, so that it was always 48 hours old before the arch was placed.

The invert section was cured by keeping it full of water. Earth plugs placed at each manhole, at about 660-ft intervals, made this possible. This water was also used for construction. The arch gunite was cured under the backfill, which was placed to a 6-in. depth over the top of the arch immediately after shooting. This backfill was first wet down by hose, and later irrigated by running water in furrows along the trench for a 7-day period.

Reinforcing consisted of a 4 by 4-in. electric welded wire mesh of 12 gage in both directions. During the



Invert Construction—Templets Set for Neat Trim; Excavation in Foreground, Completed Invert in Rear



Completed Invert with Form Bracing and Arch Forms in Place—"Shooting" Progresses in Background

work on the experimental section the first year, 60-in. widths of this mesh were used, cut long enough for a complete covering of the invert and arch sections, with the mesh for the arch laid back along the trench while the invert was being gunited. After the arch forms were placed, the mesh was laid over the forms and tied, using a one-square lap.

It was found difficult to make the mesh lie smoothly, especially on the curves, and when it was crimped or wrinkled, cracks would be found after curing. As an improvement in this procedure the following year, separate strips were cut for the invert and arch sections, and the two fastened together along both sides after the arch forms had been placed, allowing a two-square lap on each side. This eliminated both wrinkling and cracks.

The design and handling of the arch forms constituted the most important feature of the construction. These forms, as shown in the accompanying illustration, consisted of 10-gage galvanized iron sheets, 30 in. wide, rolled to form semicircular segments to the 48-in. inside diameter of the conduit. In placing, each 30-in. section was lapped 1 in. over the preceding forms to eliminate cracks and roughness in the finished surface. During the first operations, strips of cheap building paper were used over the metal forms to keep the gunite from adhering to the metal, but experiments indicated that brushing the metal with waste oil was effective at a much smaller cost.

In removing the forms, a small 4-wheel buggy was used, with ropes attached to both ends. One man inside the conduit knocked down and loaded the forms on the buggy, and one man on the outside pulled the buggy out, unloaded the forms, and stacked them along the adjacent section. A crew of only three men could remove and reset 330 ft of arch forms in one day. The same three men placed the wire mesh ahead of the shooting the following day.

Incidentally, no nails are used in this supporting system, except in the ridgepole at each post. These nails extend about $^1/_2$ in. through the 2 by 4-in. ridgepole into the posts, and were used only as tacks to keep the ridgepole in place while the metal forms were being adjusted.

After three years of service, very few cracks were found in this conduit. These were chipped out with a small portable electric chipper, and refilled with a non-hardening watertight filler.

Irrigation service was started from the completed sections of the line before the entire job was completed. In one case, the man in charge made a mistake, and instead of using the gunite line as a gravity conduit, allowed the water to back up against the bulkhead, thus placing the pipe under a 6-ft head above the top of the arch. There was no backfill to assist in holding down the arch, and the gunite was only 7 days old; but it stood up satisfactorily.

There were three reasons for constructing the conduit with a 4-ft minimum backfill: (1) to insure a uniform temperature as nearly as possible, thus reducing concrete expansion and contraction to a minimum; (2) to obtain more uniform soil moisture, thus reducing danger of soil expansion and contraction; (3) because the land is occasionally plowed to a depth of 3 ft.

The writer served as chief engineer for the Irvine Company and wishes to give credit to M. C. Cummings, construction superintendent, for working out the detail of the form bracing.

Side Spans of Suspension Bridges Under Static Lateral Forces

By Louis Balog

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It is characteristic of every statically indeterminate system that a reciprocal limitation of the deformations exists between its parts. The result is that the more rigid members of the system receive a larger share of the external loads than the statics of simple systems would suggest. A good example of this is the suspension bridge under the action of horizontal lateral static forces. Assuming these forces to be in proportion to the vertical projection of the structure, as static wind forces are, the wind truss will receive a much larger pressure than the cables. This pressure creates a transverse horizontal deflection of the wind truss during which load redistribution occurs. The redistribution is a function of the relative stiffness of the cables, wind truss, and suspenders. The stiffness of the wind truss is expressed by its transverse moment of inertia, that of the cables and suspenders by the cable and suspender pull, respectively. The span length, the dead and live load, and the suspender length also affect the relative stiffness of these parts of the structure.

To find the internal stresses in a suspension bridge under the action of horizontal static wind forces, it is necessary to determine the law of the wind-load transfer between the wind truss and the cables. Rigorous methods for such investigations have been presented. The method of successive approximations (see Leon S. Moisseiff and Frederick Lienhard, Transactions, Vol. 98, 1933, p. 1080) or that of algebraic solution (see Charles A. Ellis, *ibid.*, p. 1097) is laborious. For the majority of practical cases a satisfactory approximate solution for the center span is available (see E. L. Pavlo, *ibid.*, p. 1107). In the following an approximate method for the side spans will be derived.

In the side span the wind transfer is zero at the towers and increases along the span to a point near the end support of the truss, where it becomes zero again. The rapid drop of the wind transfer at the end of the truss has a negligible effect on the moments and deflections. The flat curve of the side-span cables, the variation of the suspender length, and the horizontal deflections of the cables and truss connected by the suspenders suggest a nearly lineal variation of the wind transfer. It will

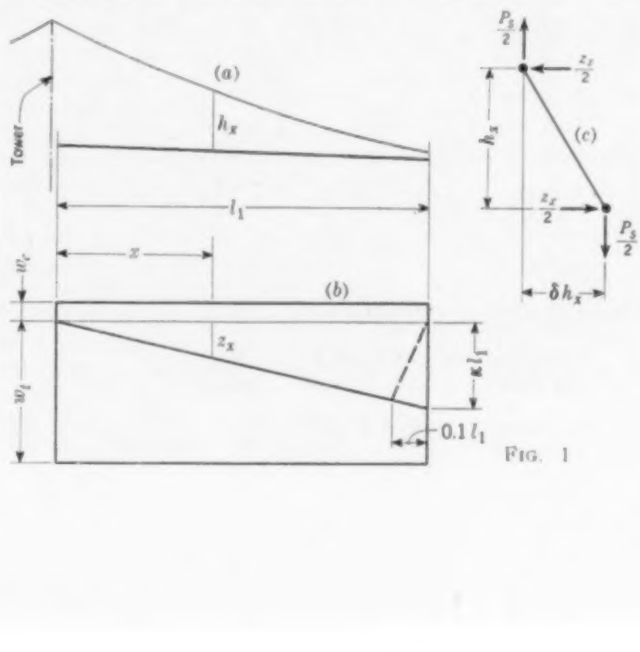

Fig. 1

be assumed that the wind transferred from the truss to the cable is a lineal function of the distance from the tower. This assumption yields quickly soluble closed expressions for the stresses and deflections and nearly the same results as does a rigorous investigation. Let:

l_1 = length of side span
H = horizontal cable pull due to vertical loads
I = moment of inertia of the wind truss
w_t = horizontal static wind pressure on suspended structure and live load per lin ft of span
w_c = horizontal static wind pressure on cables, per lin ft of span
P_s = sum of dead load of suspended structure and live load on it, per lin ft of span
z_x = transferred wind load from trusses to cables, per lin ft of span at any section, x

The assumed variation of the wind transfer is expressed by the equation:

$$z_x = \kappa x \qquad (1)$$

The pressure sustained by the wind-truss is then $w_t - z_x = w_t - \kappa x$. Both the cables and the wind-truss receive a load of trapezoidal distribution, as Fig. 1 (b) indicates.

Because of loads of such distribution, the deflection of the wind truss at $x = 0.5l_1$ is

$$\delta_{t,\,0.5l_1} = \frac{5w_t l_1^4}{384\,EI} - \frac{5\kappa l_1^5}{768\,EI} \qquad (2)$$

and that of the cable at $x = 0.5l_1$ is

$$\delta_{c,\,0.5l_1} = \frac{w_c l_1^2}{8H} + \frac{\kappa l_1^3}{16H} \qquad (3)$$

The deflection of the hanger, as seen from Fig. 1 (c), is $\delta_{hx} = \frac{h_x}{p_s} z_x$, and at $0.5l_1$,

$$\delta_{h,\,0.5l_1} = \frac{h_{0.5l_1}\,\kappa l_1}{2p_s} \qquad (4)$$

Equating the deflections $\delta_{t,\,0.5l_1} = \delta_{c,\,0.5l_1} + \delta_{h,\,0.5l_1}$ and substituting the expressions given by Eqs. 2, 3, and 4, and solving for κ, the parameter of the transferred wind load becomes

$$\kappa = \frac{\dfrac{5w_t l_1^2}{384\,EI} - \dfrac{w_c}{8H}}{\dfrac{l_1}{16H} + \dfrac{5l_1^3}{768\,EI} + \dfrac{h_{0.5l_1}}{2p_s l_1}} = \frac{N'_1 - N'_2}{D'_1 + D'_2 + D'_3} = \frac{\Sigma N'}{\Sigma D'} \qquad (5)$$

With the knowledge of κ, for the moments, shears, and deflections, the following expressions can be written:

The moment of the wind truss at any section x, is

$$M_x = \frac{w_t l_1^2}{2}\left(\frac{x}{l_1} - \frac{x^2}{l_1^2}\right) - \frac{\kappa l_1^3}{6}\left(\frac{x}{l_1} - \frac{x^3}{l_1^3}\right) = \frac{w_t l_1^2}{2} c_r - \frac{\kappa l_1^3}{6} c_t \qquad (6)$$

The shear of the wind truss at any section x, is

$$S_x = \frac{w_t l_1}{2}\left(1 - 2\frac{x}{l_1}\right) - \frac{\kappa l_1^2}{6}\left(1 - 3\frac{x^2}{l_1^2}\right) = \frac{w_t l_1}{2} c_{sr} - \frac{\kappa l_1^2}{6} c_{st} \qquad (7)$$

The deflection of the wind truss at any section x, is

$$\delta_{tx} = \frac{w_t l_1^4}{24\,EI}\left(\frac{x}{l_1} - 2\frac{x^3}{l_1^3} + \frac{x^4}{l_1^4}\right) - \frac{\kappa l_1^5}{360\,EI}\left(7\frac{x}{l_1} - 10\frac{x^3}{l_1^3} + 3\frac{x^5}{l_1^5}\right) = \frac{w_t l_1^4}{24\,EI} c_{\delta r} - \frac{\kappa l_1^5}{360\,EI} c_{\delta t} \qquad (8)$$

The deflection of the cable at any section x, is

$$\delta_{cx} = \frac{w_c l_1^2}{2H}\left(\frac{x}{l_1} - \frac{x^2}{l_1^2}\right) + \frac{\kappa l_1^3}{6H}\left(\frac{x}{l_1} - \frac{x^3}{l_1^3}\right) = \frac{w_c l_1^2}{2H} c_r + \frac{\kappa l_1^3}{6H} c_t \qquad (9)$$

In the foregoing expressions the drop of the load transfer at the end of the truss was neglected. This has little effect on the accuracy of the results except for the reaction at $x = l_1$, which will be corrected by the addition of $0.05\,\kappa l_1^2$. Assuming that the load transfer begins to drop at $x = 0.9l_1$ and diminishes in lineal variation to zero at $x = l_1$, as indicated in Fig. 1 (b), a better agreement with the actual conditions will be obtained. Leaving the value of κ the same as given by Eq. 5, the expressions for the moment and shear of the wind-truss become

$$M_x = \frac{w_t l_1^2}{2}\left(\frac{x}{l_1} - \frac{x^2}{l_1^2}\right) - \frac{0.9\kappa l_1^3}{6}\left(1.1\frac{x}{l_1} - \frac{1}{0.9}\frac{x^3}{l_1^3}\right) = \frac{w_t l_1^2}{2} c_r - \frac{0.9\kappa l_1^3}{6} c_{0.9t} \qquad (10)$$

and

$$S_x = \frac{w_t l_1}{2}\left(1 - 2\frac{x}{l_1}\right) - \frac{0.9\kappa l_1^2}{6}\left(1.1 - \frac{3}{0.9}\frac{x^2}{l_1^2}\right) = \frac{w_t l_1}{2} c_{sr} - \frac{0.9\kappa l_1^2}{6} c_{0.9st} \qquad (11)$$

Table I gives the values of the constants c_r, c_t, $c_{0.9t}$, c_{sr}, c_{st}, $c_{0.9st}$, $c_{\delta r}$ and $c_{\delta t}$.

An example that has been worked out by a rigorous method and by the method outlined in the foregoing will demonstrate the accuracy of the proposed procedure. Let

TABLE I. VALUES OF CONSTANTS

$\frac{x}{l_1}$	c_r	c_t	$c_{0.9t}$	c_{sr}	c_{st}	$c_{0.9st}$	$c_{\delta r}$	$c_{\delta t}$
0.0	0.00	0.000	0.0000	1.0	1.00	1.1000	0.0000	0.00000
0.1	0.09	0.099	0.1089	0.8	0.97	1.0667	0.0981	0.69047
0.2	0.16	0.192	0.2111	0.6	0.88	0.9667	0.1856	1.32004
0.3	0.21	0.273	0.3000	0.4	0.73	0.8000	0.2541	1.83724
0.4	0.24	0.336	0.3689	0.2	0.52	0.5667	0.2976	2.19028
0.5	0.25	0.375	0.4111	0.0	0.25	0.2667	0.3125	2.34375
0.6	0.24	0.384	0.4200	−0.2	−0.08	−0.1000	0.2976	2.27372
0.7	0.21	0.357	0.3889	−0.4	−0.47	−0.5333	0.2541	1.97420
0.8	0.16	0.288	0.3111	−0.6	−0.92	−1.0333	0.1856	1.46396
0.9	0.09	0.171	0.1800	−0.8	−1.43	−1.6000	0.0981	0.78108
1.0	0.00	0.000	0.0000	−1.0	−2.00	−1.9000	0.0000	0.00000

$l_1 = 1{,}100$ ft, $H = 29{,}400$ kips, $I = 90{,}000$ in.^{2} ft^2, $E = 29{,}000$ kip in.$^{-2}$

$w_t = 0.56$ kip ft^{-1}, $w_c = 0.06$ kip ft^{-1}, $h_{0.5l_1} = 93.65$ ft, $p_s = 7.0$ kip ft^{-1}

Substituting these values in Eq. 5, the following quantities are obtained:

$$\begin{aligned} N'_1 &= 0.338043 \times 10^{-5}\ \text{ft}^{-1} \\ -N'_2 &= -0.025476 \times 10^{-5}\ \text{ft}^{-1} \\ \hline \Sigma N' &= 0.312567 \times 10^{-5}\ \text{ft}^{-1} \end{aligned}$$

$$\begin{aligned} D'_1 &= 0.233526 \times 10^{-2}\ \text{ft}\,k^{-1} \\ D'_2 &= 0.332001 \times 10^{-2}\ \text{ft}\,k^{-1} \\ D'_3 &= 0.608117 \times 10^{-2}\ \text{ft}\,k^{-1} \\ \hline \Sigma D' &= 1.173644 \times 10^{-2}\ \text{ft}\,k^{-1} \end{aligned}$$

$$\kappa = \frac{\Sigma N'}{\Sigma D'} = 0.266332 \times 10^{-3}\ \text{kip ft}^{-2}$$

The deflection of the wind truss at $x = 0.5l_1$ is

$$\delta_{t,\,0.5l_1} = (N'_1 - \kappa D'_2)l_1^2 = 3.024\ \text{ft}$$

The deflection of the cable at $x = 0.5l_1$ is

$$\delta_{c,\,0.5l_1} = (N'_2 + \kappa D'_1)l_1^2 = 1.060 \text{ ft}$$

By the use of Eqs. 6, 7, 8, 9, 10, 11 and Table I, the moments, shears, and deflections of the wind truss and the deflections of the cable for each tenth point of the span were computed. These values are given in Table II in comparison with those obtained by rigorous analysis.

The close agreement between the computed values in Table II demonstrates the soundness of the assumption on which the approximation is based.

If the side span is relatively short, the wind-truss very rigid, and w_e comparatively large, the value of κ (Eq. 5) may become negative. In this case load is transferred from the cable to the truss, but as a rule it would be too small to significantly affect the wind-truss stresses.

TABLE II. COMPARATIVE VALUES OF MOMENTS, SHEARS, AND DEFLECTIONS

	M_x in kip ft			S_x in kips			δ_{tx} in ft		δ_{cx} in ft	
$\frac{x}{l_1}$	By Eq. 6	By Eq. 10	Rigorous Value	By Eq. 7	By Eq. 11	Rigorous Value	By Eq. 8	Rigorous Value	By Eq. 9	Rigorous Value
0.0	0	0	0	254.29	254.83	256.97	0	0	0	0
0.1	24,643	24,701	24,879	194.30	194.84	195.58	0.970	0.986	0.309	0.302
0.2	42,864	42,983	43,248	137.54	138.08	138.94	1.828	1.857	0.582	0.570
0.3	55,019	55,196	55,446	83.99	84.53	84.55	2.490	2.527	0.806	0.792
0.4	61,461	61,696	61,848	33.67	34.21	33.73	2.899	2.940	0.970	0.957
0.5	62,545	62,841	62,866	−13.42	−12.89	−13.30	3.024	3.066	1.060	1.050
0.6	58,625	58,979	58,922	−57.31	−56.77	−56.47	2.861	2.901	1.066	1.056
0.7	50,056	50,469	50,441	−97.96	−97.42	−97.15	2.427	2.463	0.975	0.962
0.8	37,193	37,666	37,701	−135.39	−134.85	−134.98	1.763	1.791	0.775	0.758
0.9	20,389	20,921	20,908	−169.59	−169.06	−170.37	0.928	0.944	0.454	0.437
1.0	0	0	0	−216.16	−216.11	−217.78	0	0	0	0

Design of Eccentrically Loaded Concrete Columns

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CONSIDERING the various code requirements, the wide variation in quality of concrete, and numerous other elements beyond the control of the designer, it would be impractical if not impossible to prepare tables and curves for designing all concrete columns with eccentric loading that might reasonably be anticipated in general practice. However, it is possible, within certain limits, to prepare tables or charts to show the general behavior of columns and to use these results as a guide in approaching ultimate designs to take care of special conditions.

With the development and better control of concrete design and proportions, it is no longer difficult to obtain concrete having an ultimate strength of 3,000 lb per sq in. in compression. If conditions demand the use of spiral reinforcing, the column should not be constructed of concrete poorer than 3,000 lb per sq in. From these assumptions, a working stress of 1,000 lb per sq in. in concrete and a ratio for the modulus of elasticity of steel to concrete equal to 10 were adopted. A further inducement to using this working stress is that the elements for any other which the designer might adopt (assuming the ratio of the moduli to remain a constant 10) could be quickly computed by direct proportion.

AS APPLIED TO CIRCULAR COLUMNS

The method of computing the design elements of circular columns by using assumed sizes and spacing of reinforcing bars presented some difficulties. It is not always possible to obtain symmetrical spacing for an assumed exact percentage of reinforcing steel; furthermore, an unsymmetrical arrangement greatly increases the computations. The expedient of substituting a hypothetical steel cylindrical shell equivalent in area to the required percentage of reinforcing bars proved quite advantageous. Careful calculations showed the two methods to be theoretically equivalent. Further, the ratio of the radius of the reinforcing steel to the radius of the column (Fig. 1 (*a*)) varies considerably from small to large columns. To meet this contingency, the data for Fig. 2 (*a*) were computed for small columns, and that for Fig. 2 (*b*) for large columns, with the idea of interpolating for all values in between. It should be noted that Fig. 2 is theoretically correct for columns of any diameter (large or small) providing the steel is placed in exact ratio to the radius of the column as indicated on the diagrams.

The data were determined by assuming cr, the distance to the line of zero stress (Fig. 1 (*a*)), then computing the area, A, of the transformed section, and the moment of inertia, I, about the gravity axis, and finally solving for P and M in the following equations:

$$\frac{P}{A} + \frac{Mer}{I} = 1{,}000 \text{ lb per sq in.}$$

$$\frac{P}{A} - \frac{M(2r - cr - er)}{I} = 0$$

It should be noted that all values are retained as a function of the radius. With P (in terms of the radius squared) as ordinate, and M (in terms of the radius cubed) as abscissa, the curves for the limiting working stress of 1,000 lb per sq in. were plotted for several values of A_s, the area of the steel.

It should be noted that the graph of the function, from the point of concentric loading to the point of zero stress on one side of the column (with c equal to zero), follows a straight line but that thereafter, with tension over part of the column, the graph follows a reverse curve. The c values are indicated by dashed lines to aid in interpolating for intermediate values of steel areas.

To use the curves, divide the computed load by the square of the radius of the assumed column; divide the computed moment, in inch-pounds, by the cube of the

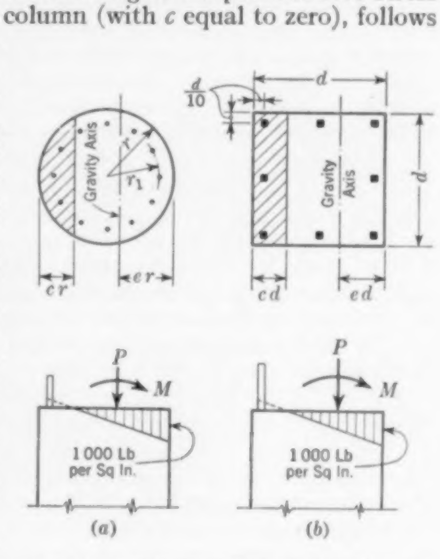

FIG. 1. ECCENTRICALLY LOADED CIRCULAR AND SQUARE COLUMNS

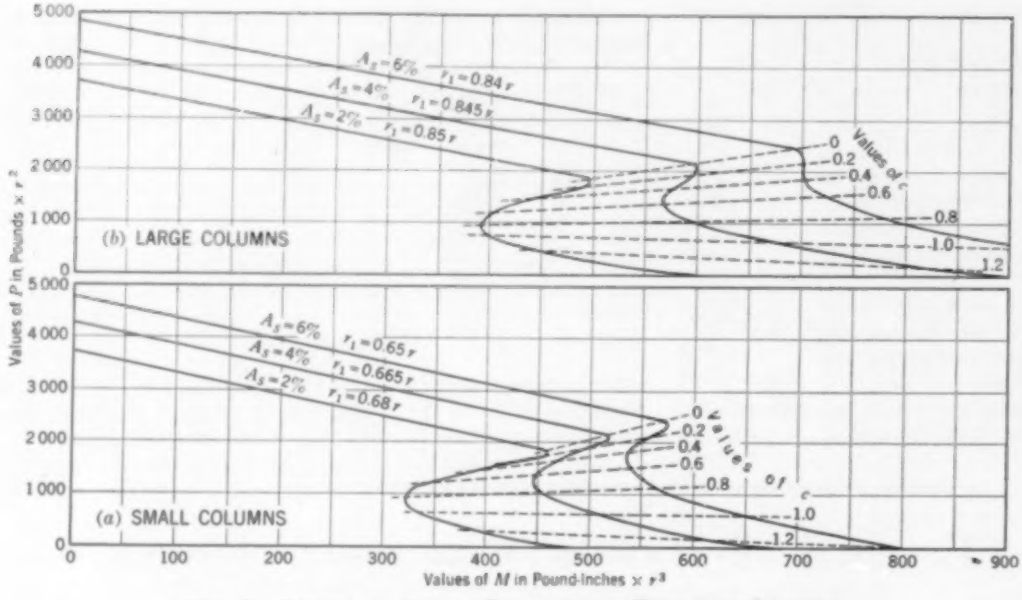

FIG. 2. DESIGN OF ROUND COLUMNS FOR ECCENTRIC LOADING

radius of the assumed column; plot this point on the graph; and read off the required percentage of steel. For example, assume a column subjected to a 38,000-lb direct load and 165,000 in.-lb of moment. Assume a column 13 in. in diameter. By slide-rule calculations:

$$\frac{P}{r^2} = 38{,}000 \div (6.5)^2 = 900$$

$$\frac{M}{r^3} = 165{,}000 \div (6.5)^3 = 600$$

From Fig. 2, the value of A_s required is 6%.

In order to determine the most economical column, capable of resisting a 38,000-lb direct load and 165,000 in.-lb of moment, make the following investigation:

ASSUMED COL.	P	M	A_s REQUIRED
13 in.	$900r^2$	$600r^3$	6%
14 in.	$775r^2$	$480r^3$	4%
16 in.	$595r^2$	$320r^3$	2%

By applying the local cost to steel, concrete, and form work, the most economical diameter of column can readily be ascertained.

It should be observed with emphasis that under certain conditions where a column is loaded to its unit working stress it can become overstressed by simply removing a part of the direct load. As an example, take the case in Fig. 2 where c equals zero and A_s equals 2%. From the diagram, the load conditions which produce a working stress of 1,000 lb per sq in. are P equal to $1{,}850r^2$, and M equal to $460r^3$. Now assume that most of the direct load P emanates from a floor above in a system of continuous frames such that a reduction in load P will not affect the value of M at the floor in question. Accordingly, suppose we reduce P to $1{,}330r^2$, leaving M equal to $460r^3$, and plot this point on the graph in Fig. 2 for the column reinforced with 2% of steel. It is found that this point falls far to the right of the curve, and the obvious conclusion is that in these circumstances the column is overstressed. As a matter of fact a rigorous analysis of this condition reveals that the column stress exceeds the assumed working stress of 1,000 lb per sq in. by 42%.

AS APPLIED TO SQUARE COLUMNS

The discussion would not be complete without a study of columns square in cross section. Following like assumptions for the circular columns, data and curves for square columns are presented in Figs. 1 (b) and 3. Since the elements for square columns can be computed with relatively little difficulty, the data assume one particular case of the steel embedded at one-tenth the depth of the column. Preliminary designs can be made quickly and altered readily to meet the special conditions confronting the designer, with the curves of Fig. 3 as a guide, and based on the following formulas:

$$\frac{P}{A} + \frac{Med}{I} = 1{,}000 \text{ lb per sq in.}$$

$$\frac{P}{A} - \frac{M(d - cd - ed)}{I} = 0$$

The horizontal break in the curves where c equals 0.1 results from the transition of the outside layer of steel from compression to tension—a transition where the ratio $n - 1$ changes to n.

It is interesting to note that a column with 1% of steel loaded in such a manner as to make c equal to zero will support approximately the same direct load and moment as a column of the same size reinforced with 2% of steel.

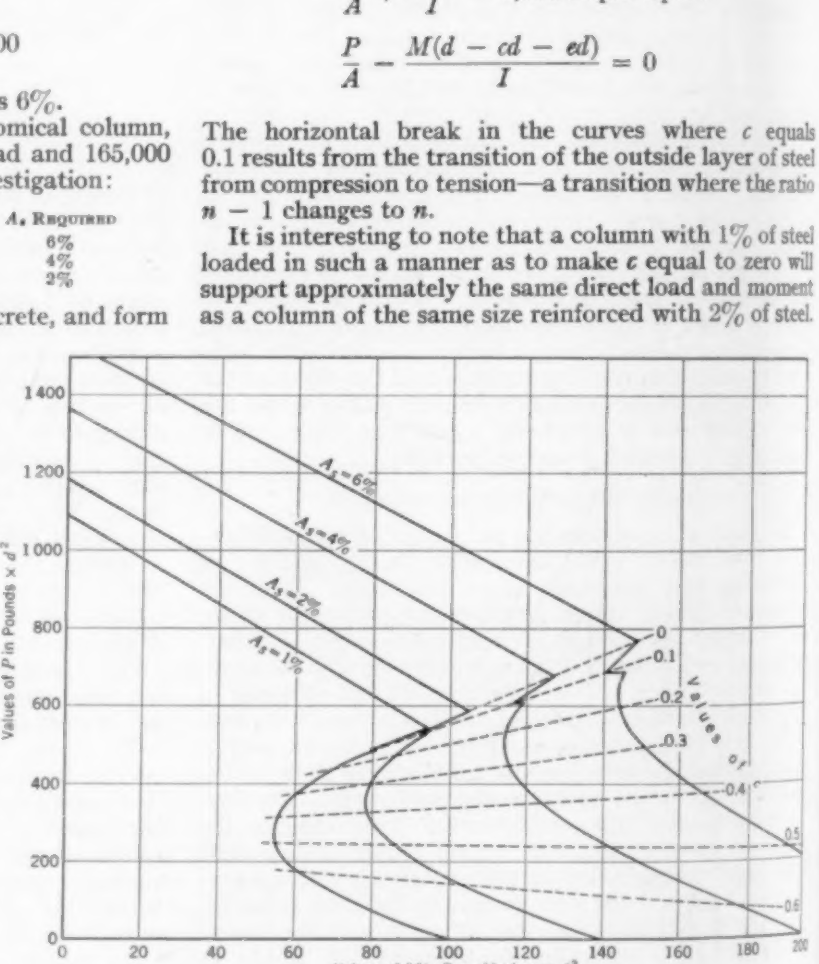

FIG. 3. DESIGN OF SQUARE COLUMNS FOR ECCENTRIC LOADING

OUR READERS SAY—

In Comment on Papers, Society Affairs, and Related Professional Interests

Trends and Development of TVA Power Program

To the Editor: In connection with Mr. Wiersema's article on the TVA power program, in the February issue, it may be of interest to compare TVA accomplishments during the past few years with estimates of cost and available power made before completion of the work.

In Table I, a comparison has been made between the actual construction cost of the six completed dams and the estimates of cost originally presented to Congress at the time authorization for construction was obtained. Proper allowance has been made for subsequently authorized changes in the scope of the work, such as additions to the generating equipment and elimination of items not included in the construction cost estimates.

Table I. Comparison of Estimated and Actual Construction Costs of Six TVA Dams

Project	First Estimated Cost*	Adjusted Estimate	Construction Cost†	Underrun
Norris	$ 36,025,230	$ 31,294,930	$ 30,754,363	$ 540,567
Wheeler	32,116,537	33,064,853	31,855,307	1,209,546
Pickwick Landing	32,529,685	30,670,316	29,793,189	877,127
Guntersville	29,500,000	35,631,000	31,098,760	4,532,240
Chickamauga	31,650,000	38,110,000	34,713,000	3,397,000
Hiwassee	15,250,000	18,601,000	16,982,000	1,619,000
Totals	$177,071,452	$187,372,099	$175,196,619	$12,175,480

* From the report to Congress on "The Unified Development of the Tennessee River System," March 1936.

† From Appendix F, "Cost Allocation Report—Annual Report of the Tennessee Valley Authority, 1940."

It will be noted that for the last three dams the difference between construction costs and estimated costs is somewhat greater than for the first three projects, and this probably reflects to some extent the greater economy made possible by such factors as the re-use of construction equipment and the employment of substantially the same engineering and construction personnel.

With regard to estimates of available primary power, the low-flow period, which extended from July 1939 to February 1940, has provided a severe test of the power estimates made prior to that time. The original estimates of available hydro power were based on stream flow conditions prevailing in 1925. During the 1939–1940 dry period various additions to the TVA power system were made, and hydro power estimates for the duration of this dry period, based on the 1925 stream flow, are shown in Table II.

Table II. Hydro Power Estimates for 1939–1940 Dry Period

Period	TVA System	Available Primary Power (Continuous Kw)
July 15–31	Norris, Wheeler, Wilson, and Pickwick Landing	274,000
Aug. 1–15	Add Guntersville Reservoir and one unit	305,000
Aug. 16–Oct. 12	Add Hales Bar, Great Falls, Blue Ridge, Ocoee No. 1, and Ocoee No. 2	352,000
Oct. 13–Feb. 4	Add Guntersville 2d and 3d units	374,000

According to the estimates in Table II the total primary hydro energy for the duration of this period would amount to 1,718,000,000 kwhr. Actual generation at the hydro plants during the period amounted to about 1,589,400,000 kwhr, with 20,000,000 kwhr remaining in storage at the end of the dry period, which was started with a hydro-energy storage deficiency of 95,000,000 kwhr because of limitations in transmission facilities prior to the acquisition of the Tennessee Electric Power Company properties. Total hydro power, which could have been generated under normal conditions in 1939–1940, would therefore amount to 1,705,000,000 kwhr as compared with the earlier estimate of 1,718,000,000 kwhr based on the 1925 stream flow year. Similar comparisons could be made between estimates and actual performance for the navigation and flood control purposes of the TVA.

The 30 projects of the 1945 TVA system as now authorized will consist of 6 steam plants, with an installed capacity of about 476,000 kw, and 24 hydro plants. The hydro plants are well diversified as to type and consist of 10 storage plants on the tributaries with a usable storage capacity of more than 8,500,000 acre-feet, four run-of-river plants on the tributaries, nine main Tennessee River plants with about 6,700,000 acre-feet of useful storage capacity, and one plant on the Cumberland watershed.

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Tennessee Valley Authority

Knoxville, Tenn.

Unusual Sources of Flood Data

Dear Sir: A few concrete examples will illustrate the truth of Mr. Larson's excellent advice, in the March issue, regarding the value of diligence and thoroughness in searching to extend the range of hydrologic records.

In our experience in the Tennessee Valley and elsewhere we have found it both desirable and feasible to develop a comprehensive flood history of each major stream for periods extending back often several times that of the recorded data. Among the valuable sources of data have been old diaries and family Bibles which were kept by the early settlers of the region. On the Holston River, for example, an old diary was located which described the floods that occurred during the eighteen sixties. The diary gave considerable information on rainfall and stream heights at a time when there were no actual rainfall or stream flow stations in the region. From the information in this old diary, it has been possible to reconstruct with reasonable accuracy both the rainfall pattern and flood hydrograph during the great flood of March 1867. Family Bibles have been found to contain valuable notes regarding floods and weather conditions. For instance, there may be a reference to the water's being over a certain field or other landmark for which it is possible to determine the elevation.

Accounts of such early travelers as Bishop Asbury of the Methodist Church sometimes tell of floods which they encountered. The Moravians who settled in North Carolina in the middle of the eighteenth century have left a valuable series of diaries which are being translated progressively. These diaries, now available in English for the period 1752–1792, contain many references to floods and weather conditions in the particular region where the Moravians lived.

The great flood of July 1916 on the Swannanoa River near Asheville, N.C., is fresh in the memory of those now living. However, research into the flood history of this stream disclosed that a flood about 5 ft higher than that of 1916 had occurred in 1791. One of four references substantiating the early flood was an old journal, which described a trip made through the Swannanoa Valley in 1800 and commented on a terrible flood that had occurred nine years before.

In the part of the Tennessee Valley which was the original home of the Cherokee Indians, information on floods has been obtained by interviewing old Indians to learn of any facts that may have been handed down from their ancestors. In a source that would be least expected to yield flood data—the "Annual Report of the Bureau of Ethnology, 1890–1891"—reference was found to a flood of unprecedented height on the Clinch River. This flood, which occurred in April 1886, swept several feet of soil from an Indian burial ground.

So many times we have discovered valuable information in the most unexpected places that we have come to regard almost any document or publication as possible source data. One would scarcely suspect that the State Board of Health Bulletin of Tennes-

see would contain rainfall and flood data for the last part of the nineteenth century, but there it is mixed in with disease and mortality statistics. Also, valuable information on stream-gage heights fifty years ago was unearthed in a dark corner of storage space in a local Weather Bureau office. Thus research carried out with systematic and painstaking diligence yields results in hydrologic information that is of real value in the planning and operation of water-control projects.

Incidentally, a "List of Climatological Records in the National Archives" has just been issued by the National Archives. Engineers interested in Mr. Larson's article will find this new publication of value.

ALBERT S. FRY, M. Am. Soc. C.E.
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Tennessee Valley Authority

Knoxville, Tenn.

Comments on Live-Load Reduction Formula for Buildings

DEAR SIR: The article on "A Proposed Live-Load Reduction Formula for Buildings" by C. W. Barber, in the April issue, gives a clear explanation of how the formula may be manipulated in practice, but it does not go sufficiently into the properly antecedent question of whether the family of curves represented by the formula pass through only acceptable points. It is to be hoped that this formula will not reach any status approaching that of an "American Standard" until code-writing bodies and other officials responsible for public safety shall, in representative numbers, have discussed the notable decreases in live load which this formula countenances for some frequently occurring cases.

A first look at the curves in Mr. Barber's Fig. 1 creates surprise that they are so steep near the origin; suspicion attaches to so rapid a reduction in unit loading for only 200, 300, or 400 sq ft of area. A test of the formula reveals that only the primary floor beams in any structure are likely to be figured for the full code live load. See Table I.

TABLE I

UNIT LIVE LOAD IN LB	MAXIMUM AREA IN SQ FT (FOR UNREDUCED UNIT LIVE LOAD)	BECOMES 1/2 LESS AT (SQ FT)
40	75	150
60	80	168
80	85	193
100	90	225
125	98	285
150	108	390

A few specific cases will now be calculated. In each case it is assumed that the construction is symmetrical, so that the design area a on the beam or girder is one-half of the tributary loaded area A.

1. An apartment living room 12 ft 6 in. by 16 ft, or 200 sq ft in area, has a transverse girder 12 ft 6 in. long at its center. The live load on this girder, under a 40-lb code, will be $100 \times 13.3 + \frac{1}{2} \times 2{,}000 - 2{,}330$ lb = 23.3 lb per sq ft = 58% of 40 lb. The author holds that this is a commendable reduction; the writer leaves it for the consideration of engineers familiar with tea-dance practice in apartment communities.

2. In another apartment room a girder supports an area 8 by 15 ft or 6 by 20 ft, equalling 120 sq ft. The live load on this girder, under a 40-lb code, would be: $w' + \frac{K}{A} = 13.3 + \frac{2{,}000}{240} = 21.7$ lb per sq ft instead of 40.

3. In an office building designed for a 50-lb live load, a girder supporting 165 sq ft (say, $10 \times 16\frac{1}{2}$ ft) would be designed for one-half less, or 25 lb per sq ft. But there are office buildings which have to be kept under constant survey to prevent accumulations of books, records, and file prints from overloading not only the beams but the girders.

4. A theater-entrance girder supporting 500 sq ft (say, 12 ft 6 in. $\times$ 40 ft) under a present code requirement of 100 lb per sq ft would be designed for $500 \times 44.5 + \frac{1}{2} \times 5{,}000 = 24{,}750$ lb = 49.5 lb per sq ft = $49\frac{1}{2}$% of code live load. But if 100 lb per sq ft can be gathered onto any part of a theater entrance it can be gathered onto the full area of such a girder.

5. A warehouse girder supporting 20 ft $\times$ 36 ft = 720 sq ft and posted for a 200-lb live load, would be designed for $720 \times 126.0 + \frac{1}{2} \times 10{,}000 = 95{,}720$ lb = 133 lb per sq ft = $66\frac{1}{2}$% of 200. But the entire floor, less walking space, may be loaded with the same load from end to end, and require policing to hold it to 200.

While the writer might prefer to see this family of curves less upright than they are, in the area between 0 and 500 sq ft, his present purpose is not to debate either curves or formula, but to emphasize that first of all an agreement must be reached as to what are proper live loads for various specific members such as those examined above. If they are found by consensus of opinion to be those loads which fit the author's formula, then of course building engineers have been quite extravagant in the design of girders and trusses. Whatever loads may be agreed upon, to fit the different members and different circumstances that should be considered, then it may be possible by ingenuity such as the author has shown, to fit a curve thereto. But the writer fears that a single formula will prove to be too generalized, and that some exceptions, in addition to the present exception of roof loads, will prove advisable.

This topic has been brought up concurrently with a widespread discussion of increasing the unit stresses; the two topics are parts of one problem.

JONATHAN JONES, M. Am. Soc. C.E.
Chairman, Committee on Specifications,
American Institute of Steel Construction

Bethlehem, Pa.

Use of the Extensometer in Making Accurate Measurements

TO THE EDITOR: I read with much interest the article by Prof. P. E. Soneson, entitled "Measured Stresses in a Two-Hinged Beam Arch" in the April issue of CIVIL ENGINEERING. The subject of measured strains is one of particular interest to all engineers engaged in the design or construction of buildings or bridges.

A few of the writer's experiences in connection with the preparation of the gage holes in members to be measured, and the application of the extensometer for accurate measurements, might profitably be mentioned. Some years ago the writer became thoroughly grounded in the importance of precision in such work during a comprehensive investigation on important truss members at the main towers of the Williamsburg Bridge and the legs of the Brooklyn intermediate towers. This investigation was conducted by the New York City Department of Bridges, in cooperation with the National Bureau of Standards.

A high degree of accuracy should be observed in drilling and reaming the gage holes, which must be normal to the plane of the member. However, the holes drilled and reamed in the bridge members are rarely so well made. The holes should be smoothed with a conical set after drilling and reaming, and they must be cleaned after each reading. Otherwise, dust or grit remain in them and interfere with the precision of the measurements.

In taking accurate gage readings, the extensometer must be held in a normal position (provided the drilled and reamed holes are strictly normal to the plane of the work), using the same end of the gage over the same holes in each measurement, and the pressure applied should be directly over the contact point. In the hands of those skilled in the making of refined measurements, no errors of computation, fabrication, or erection, could reasonably escape attention.

In other words, from the writer's personal experience with the use of the Howard extensometer for precise work, persistent patience and painstaking thoroughness, combined with good judgment on the part of the observer, are required until he gains perfect control of the instrument. After that the observer will experience little or no difficulty. It must be remembered, however, that it is upon the observer that the measurements will depend, and the skill required to obtain reliable results is not in the possession of all. The Howard extensometer, it should be added, was designed by the late James E. Howard, engineer physicist for the National Bureau of Standards, and used by him exclusively for stress investigations in the Bureau.

THEODORE BELZNER, Affiliate, Am. Soc. C.E.

Brooklyn, N.Y.

SOCIETY AFFAIRS

Official and Semi-Official

Spring and Summer Society Meetings

IN AN ATMOSPHERE of patriotism, attuned to the war effort, the Spring Meeting of the Society convened at the Hotel Roanoke, Roanoke, Va., April 22 and 23, 1942. Previously, on the twenty-first, a Local Section Conference was held to accommodate delegates from the southern and eastern districts.

It will be recalled that the Roanoke Meeting was planned on the new idea that it would be run by the Society and its staff rather than by a Local Section. Thus the efforts of the local members, now so essential to war activities, need not be drafted. This made it possible to choose a location like Roanoke, which is not the center of a large group of Society members.

Meeting Stresses War Topics

The choice was a happy one. Although it was realized that the same large attendance as formerly could not be expected, nevertheless the meeting was successful in every way. A generous group of earnest Americans gathered to discuss vital problems and took away with them many ideas that would be of service, not only in individual activities, but in the efforts of the nation at large.

Following the formalities at the opening session on Wednesday morning, including the singing of "America," the remainder of the general session was given over to a consideration of highway problems in wartime. The afternoon sessions were on the subject of war protection under the significant watchword, "Prepare before instead of repair afterward." At the same time students from the surrounding Chapters were holding an all-day conference with a varied program.

Dinner on Wednesday evening was the only formal gathering of the meeting. An excellent dinner, a fine speech on post-war planning, and social dancing were the ingredients of the program.

The entire day Thursday was given over to sessions of the Technical Divisions. Airports, steel conservation, sanitation during wartime, construction of defense plants, and surveys for war use were some of the topics. It was quite fitting that in the midst of this program, the Chief of Engineers should address a general luncheon on the subject, "This Is an Engineer's War."

Enjoyable Social Events

Immediately following lunch the ladies left on an afternoon trip to Lexington, in the course of which they visited the campuses of both Washington and Lee University and Virginia Military Institute. Most of the men waited for the afternoon sessions, but immediately afterward set out by automobile to meet the ladies at the Natural Bridge of Virginia. There the whole party spent the evening following a delightful picnic supper.

In connection with the Spring Meeting a number of committees and other gatherings were held. The largest of these was the Local Section Conference on Tuesday. A large number of delegates spent the entire day in discussing matters of intense interest to the Society, particularly as regards war-time activities.

In keeping with the spirit of the times, the meeting was made short and came to an end on Thursday. The general consensus was that the new form of meeting was admirable. All agreed, also, that the choice of location, with its beautiful mountain scenery and balmy spring climate, was ideal. The attendance totaled well over three hundred.

Summer Convention Set for Minneapolis

With the conclusion of the Spring Meeting, thoughts naturally turn to plans for the summer Convention. Arrangements have now been confirmed by the Board of Direction at its Roanoke Meeting whereby the Convention will be held on the campus of the University of Minnesota at Minneapolis, on Wednesday and Thursday, July 22 and 23, 1942. This arrangement is by agreement with the local members and officers of the Northwestern Section, the University officials, and civic authorities.

Sessions will be held mostly on the campus. A splendid and spacious two-million-dollar structure, the Coffman Memorial, will house most of these sessions. This applies not only to the technical gatherings, but also to the formal dinner, usually scheduled for Wednesday evening. A number of the Technical Divisions are planning programs, the details of which will be announced in subsequent issues of CIVIL ENGINEERING.

Ample accommodations are available, not only for the meetings but for housing. Many of the men will doubtless stay on the campus in dormitories. City hotels, within street-car distance, are also available for those who may desire this type of accommodation. It is understood that plans for the summer school and for military training at the University will not interfere with the Society's Convention.

Intellectual and Other Attractions

The papers presented will center on the general topic of ways and means for engineers to assist the Government in efforts to win the war. It happens that the time of the Convention coincides with that of a celebrated event called the "Aquatennial." This is a festival extending throughout the week, with colorful parades, rodeos, and regattas. The war scene during the current year will be emphasized.

Among other features of interest will be the large and up-to-date plant of the University of Minnesota. It is beautifully situated near the Mississippi River, with many buildings and a fine campus It comprises an intellectual center for 14,000 students normally. Among the special attractions for engineers is the new Hydraulics Laboratory at St. Anthony Falls. This is close to the campus and hence convenient for the meeting. It probably will be utilized for sessions having to do with hydraulics. Equipment is available for large-scale hydraulic experimentation as well as for model testing.

All in all the choice of the University of Minnesota for this meeting should meet with hearty approval. The date is one to be remembered—July 22 and 23. Many members and other engineers, not only from the Upper Mississippi Valley but from surrounding areas, will wish to make their plans accordingly.

WHERE SOME SESSIONS OF THE ANNUAL CONVENTION WILL BE HELD
Coffman Memorial Union at University of Minnesota, Minneapolis, Minn.

Arthur N. Talbot, Past-President and Honorary Member, Dies

Members of the Society will be grieved to hear of the death—in Chicago on April 3—of Arthur Newell Talbot, Past-President and Honorary Member. Dr. Talbot was born at Cortland, Ill., on October 21, 1857. He graduated from the University of Illinois in 1881, with the B.S. degree, and four years later received the degree of C.E. Following his graduation, he engaged in railroad work, having been with the Denver and Rio Grande Railroad, the Atchison, Topeka and Santa Fe, and the Northern Pacific.

From 1885 to 1926 he was a member of the engineering faculty at the University of Illinois—from 1885 to 1890 as assistant professor of engineering and mathematics, and from 1890 to 1926 as professor of municipal and sanitary engineering in charge of theoretical and applied mechanics. Coincidentally for part of this period (1895 to 1930) he acted as consultant on various engineering projects. Although he retired in 1926, with the title of professor emeritus of engineering, he continued his research.

Arthur Newell Talbot

He did pioneer work in the properties of concrete, reinforced concrete, and other structural materials, and his research into track stresses led to the modern right-of-way. In 1903 Dr. Talbot helped found the University of Illinois Engineering Experiment Station, the first of its kind in the country, and in 1938 the new materials testing laboratory at the University was named for him—an unusual honor for a living person.

The author of more than 400 articles and bulletins, Dr. Talbot was the recipient of many engineering honors, including the John Fritz Medal, the Washington Award, and the Lamme Medal. His engineering affiliations included honorary membership in the American Society for Testing Materials, the Western Society of Engineers, the American Railway Engineers Association, and the American Concrete Institute.

Dr. Talbot had the distinction of having served in many Society offices—he was Director in 1909 and 1910, Vice-President in 1911, and President in 1918. He was elected Honorary Member in 1925.

New Supply of Manual on Ethics Available

For the benefit particularly of graduating students in the Society's Student Chapters, note is made of the fact that there is now an adequate supply of Manual 21, on ethics, available on request to Society Headquarters.

This Manual, entitled "Standards of Professional Relations and Conduct," was prepared by Daniel W. Mead, Past-President and Honorary Member, Am. Soc. C.E. It is intended to cover, as far as practicable, the personal and ethical relationships of the engineer in all the ordinary positions and branches of the profession. It is addressed more particularly to the younger men, in the hope of supplying them with useful information as a guide to action and conduct in the practical relationships of their dawning professional life.

In ordering this Manual, members of Student Chapters may take advantage of the special rates available to Society members, as follows:

Single copies	$ .25
10 copies	2.50
25 copies	6.00
100 copies	12.50
200 copies	18.00

This Manual is a worth-while addition to the engineer's library.

Iowa Institute Announces Program of Second Hydraulics Conference

As announced in the April issue of Civil Engineering, the Institute of Hydraulic Research of the State University of Iowa will hold its Second Hydraulics Conference at Iowa City June 1–4. Under the joint sponsorship of the American Society of Civil Engineers, the American Society of Mechanical Engineers, and the Society for the Promotion of Engineering Education, the conference will emphasize principles of fluid motion of importance to a wide variety of other engineering fields and defense agencies of the Government.

For the eight sessions, as arranged to date, the following program has been set up:

Monday, June 1

Morning: Scope and Importance of Fluid Mechanics (2 papers)
Afternoon: Modern Methods of Research (4 papers)

Tuesday, June 2

Morning: Mechanics of Fluid Resistance (4 papers)
Afternoon: Inspection of Hydraulics Laboratory

Wednesday, June 3

Morning: Cavitation Phenomena (3 papers)
Afternoon: Problems of Wave Motion (4 papers)

Thursday, June 4

Morning: Engineering Aspects of Fluid Turbulence (4 papers)
Afternoon: Sediment Transportation (4 papers)

Invitations to present papers have been accepted by a particularly distinguished group of engineers and scientists, including university representatives and government officials. It is highly probable that this second conference will attract as representative a gathering of engineers as attended the first conference in 1939.

Dormitory and hotel accommodations will be arranged by the Institute for those attending the sessions, and in addition one of the large fraternity houses on the campus will be open exclusively to conference guests. Reservations may be made in advance by writing to Prof. J. W. Howe, Engineering Building, University of Iowa, Iowa City.

Surveying Instruments Needed by Corps of Engineers

The War Department has issued an appeal to the public for surveying instruments for the use of the Corps of Engineers in construction work. Most urgently needed are transits, levels, and plane tables.

All surveying equipment in the hands of commercial firms has been purchased by the Corps of Engineers, but there is still an acute shortage of instruments for the work on hand. It is believed that many instruments of this nature are owned by individuals or firms throughout the United States, who are not using them at present and who would be glad to turn them over to the Engineers.

The Corps of Engineers is prepared to rent, purchase, or borrow these instruments, all sales to be made with or without the recapture clause, as the owner shall decree. Anyone having such instruments available is requested to write to Maj. R. L. Richardson, Office of the Chief of Engineers, Construction Division, Washington, D.C.

Changes in Officers—Soil Mechanics and Foundations Division

Developments of the war have required a realinement in the names of officers of the Soil Mechanics and Foundations Division of the Society. This is due to the fact that the former chairman, Carlton S. Proctor, has recently become a colonel in the Engineer Corps, and is expecting to be sent on overseas duty; and the secretary, Charles F. Travis, has been transferred to Venezuela, on a more or less permanent assignment. In their places, Joel

D. Justin, M. Am. Soc. C.E., has been assigned to the chairmanship of the Division, and Hamilton Gray, Jun. Am. Soc. C.E., has been appointed by the executive committee as secretary. Both men are well versed in this subject—Mr. Justin has been for many years a prominent consultant in Philadelphia, and Dr. Gray is an assistant professor at New York University.

Daniel W. Mead Prize Competition

"TIME marches on," and so does the third annual competition for the Daniel W. Mead Prizes offered for the best student and Junior papers in the field of ethics. Prospective contestants are reminded of the fast approaching deadline—July 1, 1942.

Attention is drawn to the requirements for entry: student papers must have been presented before a Student Chapter, a Chapter Conference, or a Local Section, and Junior papers before a Local Section or a Local Section Conference. The rules, as they appear in the current 1942 Yearbook of the Society, remain unchanged; copies are available for distribution on request.

Again, the submission of six copies of all entries is desirable; however, inability to prepare additional copies should not prevent or delay the entry of any paper.

The student topic is "Ethical Standards and How Best They Can Be Developed"; the Junior topic is "Observations of Ethical and Unethical Practice by Older Engineers."

1942 Yearbook Issued

AS IN previous years, the current issue of the Society's Yearbook has been mailed to the membership with the April PROCEEDINGS. This marks another step in the progress and development of the Society.

Because of the steady increase in Society membership, the size of the Yearbook tends to increase year by year. This year the 17,538 members who were on the record when the list closed, necessitated enlarging the book by more than 20 pages above the size of a year ago.

A great deal of time is put into the production of the Yearbook by members of the staff at Society Headquarters. Every effort is exerted to make the information as accurate and up to date as possible. In particular it is to be noted that the Annual Report of the Board of Direction is given in this volume. This is the only place where the full report is made available to members.

Much of this report deals with the contributions of the Society in the war effort—a matter of pride to all members. With so many of the membership now on active duty with the Army and Navy, and so many others directly and indirectly engaged in war work, it is particularly helpful to have the 1942 Yearbook as a reference.

New Schedule of Dues for Juniors

DUES FOR Juniors of the Society, by amendment to the Constitution, as announced in the April issue, are now modified and have become effective at the new rates. Although actually the amendment became effective on April 17, or 30 days after the canvass of ballots, the computations of the revised dues will be based on the assumption that the new rates went into effect a few days later, or on May 1. In this way there will be one-third of the year under the old rates and two-thirds under the new rates.

For Juniors less than 32 years of age, no change is involved. For those who are 32 and older, the annual dues will be as follows:

AGE	NON-RESIDENT	RESIDENT
Up to 32	$10.00	$15.00
At 32	12.50	17.50
At 33	15.00	20.00
At 34	17.50	22.50

In determinig the new dues for Juniors, the first third of the year is computed at the old $10 or $15 rate, and the remaining two-thirds of the year at the new rates. First as to those who became 32 prior to May 1, 1942—bills will be made up and forwarded on the following basis:

JAN. 1–APRIL 30		MAY 1–DEC. 31		TOTAL 1942 DUES	
Non-Res.	Res.	Non-Res.	Res.	Non-Res.	Res.
$3.33	$5.00	$8.33	$11.66	$11.66	$16.66

If a Junior reaches age 32 after May 1, the two components will be proportioned as of the first of the month next following his birthday.

Pending the operation of the amendment, some Juniors have been retained in membership beyond the normal age limit and have reached their thirty-third birthday. For these Juniors, a similar computation, by a combination of old and new rates, yields the following dues for 1942:

JAN. 1–APRIL 30		MAY 1–DEC. 31		TOTAL 1942 DUES	
Non-Res.	Res.	Non-Res.	Res.	Non-Res.	Res.
$3.33	$5.00	$10.00	$13.34	$13.33	$18.34

If age 33 is reached after May 1, 1942, the calculation is somewhat more complicated and will consist of the first four months at the old rate, the next months to and including the month of the birthday at the rate for age 32, and the remainder of the year for age 33. All Juniors will receive statements—for the whole year's dues on the new basis if they have not already paid, and for the additional payment required if they have paid.

Although it is the intention to forward a notice to all Juniors, this item will supplement such notice to make doubly sure that every Junior receives the necessary information. After the first few months, the overlapping ages will be eliminated, and thereafter the dues will be readily found from a straight computation on a yearly basis.

Special Papers Available in Library

NOT ALL of the excellent papers submitted for publication to the Society can be accommodated in the regular publications. Certain of these manuscripts, therefore, are designated by the Publications Committee for filing and reference in the Engineering Societies Library. With the approval of the authors, the following papers have been so filed. They may be consulted in the Library or they may be photostated. Quotations on this service will be furnished by the Library on request.

ELASTIC PILE WITH HORIZONTAL FORCE

STANLEY U. BENSCOTER, Jun. Am. Soc. C.E., "An Elastic Pile with Horizontal Force" (15 pages of text, double spaced, including four graphs). This is a mathematical treatment of an elastic prismatic pile embedded in granular, cohesionless material, and acted upon by horizontal force at or above the surface of the soil. The analysis gives derivation for the depth of soil near the surface that is assumed to be in a plastic state of stress, and also for the displacement.

EXAMPLE OF DEFENSE HIGHWAY

EZRA B. WHITMAN, M. Am. Soc. C.E., "National Defense Highways Through Maryland" (21 typewritten, double-spaced pages plus 2 detailed maps of the state). After reviewing some of the general considerations for defense highways, this paper explains the procedures recommended by the Government. Then it considers in some detail how these considerations affect Maryland, what method of financing might be attempted, and what revision of state roads would be attempted in various priorities. This paper was originally presented before the Society Spring Meeting in Baltimore, April 1941.

STRESSES IN CIRCULAR PLATES

R. S. CHEW, M. Am. Soc. C.E., "Analysis of Plates on Circular Supports" (28 typewritten, double-spaced foolscap pages plus 7 pages of diagrams). The paper is a mathematical treatment, intended to present solutions to the problem of determining maximum stresses for various conditions of plates on circular supports. The analysis is on the basis that a system of shears holds an elemental block of the slab in equilibrium. In addition to the mathematical treatment, reports of tests are quoted from the University of California, and from Columbia University.

MULTIPLE-PURPOSE RIVER REGULATION

STANFORD MCCASLAND, Assoc. M. Am. Soc. C.E., "Planning of Multiple-Purpose River Regulation Projects" (40 typewritten

double-spaced pages, plus one graph and one table). The paper contains a summary of the factors involved in the study of the feasibility of each of the four major items of multiple-purpose projects, namely, irrigation, flood control, power, and other benefits. Particular attention is paid to irrigation practice. The conclusion is that whatever the allocation is among the functions, the total benefit to the community must be the maximum attainable.

SOLVING DEFLECTION OF TRUSSES

R. P. V. MARQUARDSEN, "The Deflection of Trusses" (12 typewritten double-spaced pages including three tables, plus three figures). The intention is to present a short practical method for determining deflections and panel-point movements of trusses. It is partly analytical and partly graphical. It consists primarily of the computation of movements on an increased scale basis, together with the plotting of a picture diagram illustrating the movement. Results are obtained by scaling.

THE UNIVERSAL FLOW FORMULA

T. BLENCH (India), "A Universal Flow Formula for Uniform Turbulent Flow in Pipes and Channels" (31 pages of text, double spaced, foolscap size, including 11 figures). Instead of deriving a universal velocity distribution law from which flow formulas follow, after some approximation, by integration, this paper first establishes a universal flow formula from dynamical considerations and accepted data. The universal velocity distribution formulas for different states are deduced independently and found to agree in form with what the flow formulas suggest.

SANITARY PROGRESS IN OHIO

F. H. WARING, M. Am. Soc. C.E., "Development of Municipal Water Supply, Water Purification, Sewage Treatment and Stream Pollution Control in Ohio" (15 pages of text double spaced, plus 3 diagrams). This is a brief history of the development of municipal water supply, water purification, and sewage treatment in the State of Ohio. It shows the progress that has been made over a period of years, from health and other standpoints. Originally the paper was delivered before the Society's 1940 Fall Meeting in Cincinnati, Ohio.

EFFICIENCY OF DREDGING

R. L. VAUGHN, M. Am. Soc. C.E., "Economics of Dredge Pumping" (29 pages of text, double spaced, plus 8 diagrams). After discussing the general aspects of dredging economics, the paper purposes to present a means of determining the dredge discharge-pipe velocity at which the dredging cost per cubic yard will be a minimum. This is to be based on actual experience with the particular dredge on the same or a similar site. With such knowledge of actual output and subdivided unit costs for a given discharge velocity, an estimate is made of output at other velocities, and corresponding unit costs are computed.

COST OF CLEVELAND SEWAGE IMPROVEMENT

J. E. A. LINDERS, M. Am. Soc. C.E., "Cost Analysis of Improvements to Sewage Treatment Facilities, Cleveland, Cuyahoga County, Ohio" (32 typewritten double-spaced foolscap pages, plus 5 maps, plans, and diagrams). After brief description of three large new sewage works for Cleveland, Ohio, this paper records a large number of data on contracts, rates, quantities, and amounts. The purpose is to furnish engineering information from the analysis of construction and engineering costs covering approximately $8^1/_2$ million dollars worth of construction work.

EARTHQUAKE INVESTIGATIONS

HEINRICH NEUMANN (Palestine), "The Effect of Earthquakes on Buildings" (17 pages of text, single spaced, foolscap size plus 7 photos). Interesting interpretations are given of the effect of earthquake motions on structures of various types. For its understanding, the paper does not depend upon acquaintance with the theory of vibration. The investigation and many of the examples are based on experiences in Palestine.

RIGID-FRAME ANALYSIS

JOHN B. WILBUR, Assoc. M. Am. Soc. C.E., "Compatibility Equations for Rigid Frames" (15 typewritten, double-spaced pages, plus 3 figures). Equations are developed for checking the compatibility of deformations corresponding to solutions by the slope deflection method, for rigid frames composed of prismatic members arranged in rectangular panels. The general principles, with proper modifications, may be applied to the compatibility of distortions in other types of structures. Numerical examples are included.

THEORY OF HYDRAULIC-FILL DAMS

JOEL B. COX, Assoc. M. Am. Soc. C.E., "Design of Hydraulic-Fill Dams to Meet Construction Stresses" (39 typewritten, double-spaced pages including 12 figures, plus 8 blueprints of computations and diagrams). A somewhat mathematical treatment of the problem of clay consolidation is presented. It gives methods of mathematical analysis of laboratory tests as utilized in connection with the reconstruction of the Alexander Dam on the Island of Kauai, Hawaii, following its failure.

Death of Alex Dow, Honorary Member

ALEX DOW, internationally known utility executive and chairman of the board of the Detroit Edison Company, died at Ann Arbor, Mich., on March 22. Had he lived until April 12 he would have been 80. A native of Scotland, Mr. Dow came to this country in 1882, settling in Baltimore, where he was employed in various departments of the Baltimore and Ohio. In 1888 he joined the British Electric Company of Cleveland, and his first assignment was that of installation electrician at the company's Chicago office. During this period Mr. Dow installed the underground lighting system for Chicago's parks.

ALEX DOW

In 1893 he went to Detroit to design and supervise the construction of the city's public lighting plan. Three years later he became associated with the Edison Illuminating Company, predecessor of the Detroit Edison Company. He became president of the latter company in 1912, serving until 1940 when he was made chairman of the board. It was largely under his direction that the organization expanded into one of the foremost utilities in the country.

He was water commissioner of Detroit from 1916 to 1921 and, again, from 1925 to 1932. In the latter year Mr. Dow was appointed district chief of the Detroit Ordnance District of the War Department, serving in that capacity until his death.

He was a member and former president of the American Society of Mechanical Engineers and an honorary member of the Institution of Electrical Engineers (Great Britain) and the American Institute of Electrical Engineers. In 1936 he was the recipient of the Edison Medal of the latter organization. Mr. Dow was elected a member of the American Society of Civil Engineers in 1906 and Honorary Member in 1936.

Additional Library Service Announced

As in the past, engineers may obtain photostatic copies of periodical articles from the Engineering Societies Library—at a cost of 25 cents a page to members, and 30 cents to others. In addition to this service, the Library is now equipped to supply microfilm copies of articles. The cost of the film, which is for a 35-mm projector, is $1 for twenty frames (the minimum charge). Periodicals will be lent only when bound, and then only when the photostatic or microfilm reproductions will not answer the need.

There is no printed catalog of the Library, but inquiries as to what can be supplied on any subject will be answered gladly.

News of Local Sections

Scheduled Meetings

ALABAMA SECTION—Two-day meeting at Montgomery, Ala., May 15 and 16, beginning at 10 a.m.

ARIZONA SECTION—Spring meeting at the Pioneer Hotel on May 2, at 10 a.m.

CENTRAL OHIO SECTION—Dinner meeting jointly with the Ohio State University Student Chapter at Pomerene Hall, Ohio State University Campus, on May 12, at 6:30 p.m.

LOS ANGELES SECTION—Inspection and dinner at the California Institute of Technology on May 13, at 5 p.m. (Group will be guests of the California Institute of Technology Student Chapter.)

METROPOLITAN SECTION—Annual meeting at Columbia University on May 20, at 8 p.m.

MIAMI SECTION—Dinner meeting at the Seven Seas Restaurant on May 7, at 7 p.m.

NORTHWESTERN SECTION—Dinner meeting at the Minnesota Union on May 4, at 6:30 p.m.

PHILADELPHIA SECTION—Dinner meeting at the Engineers' Club on May 12, at 6 p.m.

SACRAMENTO SECTION—Regular luncheon meetings at the Elks Club every Tuesday, at 12:10 p.m.

SAN FRANCISCO SECTION—Dinner meeting of the Junior Forum at the Engineers' Club on May 28, at 5:30 p.m.

SEATTLE SECTION—Dinner meeting at the Engineers' Club on May 31, at 6:30 p.m.

SPOKANE SECTION—Luncheon meeting at the Davenport Hotel on May 8, at 12 m.

TACOMA SECTION—Dinner meeting at the Lakewood Community Center on May 12, at 6:30 p.m.

TEXAS SECTION—Luncheon meeting of the Dallas Branch at the Dallas Athletic Club on May 4, at 12:10 p.m.

TOLEDO SECTION—Joint dinner meeting with the Toledo University Civil Engineers at Toledo University on May 6, at 6:30 p.m.

UTAH SECTION—Dinner meeting at the Beau Brummel Restaurant on May 6, at 7 p.m.

Recent Activities

Arizona Section

The Arizona Section—with the University of Arizona and the Arizona State Highway Department—sponsored the Fifth Annual Roads and Streets Conference. This affair, which took place in Tucson on March 20 and 21, is considered one of the major activities of the Section. The list of speakers included P. F. Glendening, resident engineer for the Arizona State Highway Department; Dr. L. I. Hewes, chief of the Western region of the Public Roads Administration; George D. Whittle, senior highway bridge engineer for the Public Roads Administration; J. W. Powers and H. H. Brown, respectively, engineer of materials and assistant engineer of materials for the Arizona State Highway Department; Bernard C. Hartung, assistant maintenance engineer for the Nevada Department of Highways; G. L. McLane, senior highway engineer for the Public Roads Administration; R. M. Morton, vice-president of the American Bitumuls Company; and John C. Park, professor of highway engineering at the University of Arizona. In addition, a number of the papers were discussed. The final session consisted of a symposium on aerial photography, the participants being Donald Scott, city manager of Phoenix; A. C. Blakey, chief of the Regional Cartographic Division of the Soil Conservation Service at Albuquerque, N.Mex.; Howard B. Waha, assistant regional forester for the U. S. Forest Service at Albuquerque; R. R. Monbeck, chief of the Topographic Branch for Arizona of the U. S. Geological Survey; and George T. Grove, county engineer of Pima County, Arizona.

Cleveland Section

A talk on bridge architecture by Dr. Sara Ruth Watson was the feature of the February meeting. Dr. Watson, who is the daughter of the late Wilbur J. Watson and co-author with D. B. Steinman of a book on bridges, gave a résumé of bridge building from the early days of the semicircular arch to the present variety of structures. At the March meeting George D. Whitmore, chief of surveys of the Maps and Surveys Division of the TVA, spoke on aerial mapping. Mr. Whitmore described the procedures employed by the TVA in mapping from aerial photographs, illustrating his lecture with maps of actual work and photographs of various reductions and enlargements as required by the photogrammetric equipment. He stressed the importance of proper surveys and maps in connection with proper surveys and topographic mapping for engineering projects.

Florida Section

A joint luncheon meeting of the Section and the Engineering Professions Club of Jacksonville was held in Jacksonville on March 24. The speaker was James E. Jagger, Field Secretary of the Society, who discussed the role of the profession in the present emergency, the Vinson Bill, and the trend toward unionization of engineers. That evening Mr. Jagger was guest of honor at a "picnic steak-fry" in Gainesville, where he had the opportunity to meet the University of Florida Student Chapter members.

Georgia Section

At the March dinner meeting of the Georgia Section Prof. Alan Pope, of the Georgia School of Technology, spoke on "Air Power and National Defense." In his talk Professor Pope compared the relative speeds, carrying capacities, and fighting power of the American and foreign planes. Following a period of informal discussion, the group adjourned to attend a meeting of the Georgia Engineering Society, at which George H. Bond, Atlanta architect, reported on the National Training Conference on Aerial Bombardment.

Illinois Section

A special meeting of the Illinois Section was called in Chicago on March 20 in order to hear T. L. Condron report on the National Training Conference on Aerial Bombardment. Later there was a round-table discussion—led by W. E. Lofgren—on the activities of the city bridge department in organizing protection for the fifty-six movable bridges within the city limits. The other speakers were C. B. Burdick, S. A. Greeley, Henry Penn, J. S. Redden, F. A. Randall, and Edward Haupt.

Indiana Section

The protection of structures against aerial bombardment was discussed by Philip E. Soneson at the March meeting of the Indiana Section. Mr. Soneson, who is assistant professor of architectural engineering at Purdue University, prefaced his talk with comments on the duties of engineers in civilian defense. Engineers are urged to cooperate with local defense units and to pool their information and experience. He pointed out that the American Institute of Architects lists, in its monthly publication, a résumé of current information regarding civilian defense. Motion pictures showing the effect of bombings on various types of structures in England, China, and Belgium concluded the evening.

Los Angeles Section

Speakers at the March meeting of the Los Angeles Section were George F. Olson, manager of laboratories for the General Petroleum Corporation, whose subject was "The Position of the Chemical Engineer in Industry," and Randolph Leigh, who discussed the possibility of an enemy invasion of Southern California. Mr. Olson predicted that there will be no rationing of gasoline in the

Los Angeles–San Francisco areas, but rather that there will be a surplus, as automobile gasoline forms a necessary by-product in the refining of fuel oil and aviation gasoline required by our armed forces in enormous quantities.

Metropolitan Section

On March 10 the Section held a joint technical meeting with the New York section of the American Welding Society. A talk on welding—its growth, the difficulties encountered, and its final acceptance as a sound practice—initiated the program. This was given by John F. Willis, engineer of bridges for the Connecticut State Highway Department. Other speakers were Dr. Miller McClintock, director of the Bureau for Street Traffic Research at Yale University, and Frederick H. Frankland, director of engineering for the American Institute of Steel Construction.

Northwestern Section

"Australia Today and Tomorrow" was the subject of a timely talk at the April meeting of the Section. This was given by Albert E. Tuck, of St. Paul, who spent his early life in Australia. Walter E. Jessup, acting Assistant Secretary of the Society, then outlined some of the activities of the Society and discussed the role of the organization in the emergency.

Oregon Section

A symposium on the Bonneville Project was the feature of the March meeting of the Oregon Section. C. C. Galbraith, principal engineer for the U. S. Engineer Department, spoke on the construction of the Bonneville power plant. He was followed by R. L. Tidball and R. C. Schuknecht, who discussed the hydraulic and electric problems encountered in the design of the project. A talk on "The Use of Bonneville Power in National Defense"—given by R. M. Miller, of the Bonneville Power Administration—concluded the program.

Philadelphia Section

There was a record attendance of over 300 for the March meeting of the Section, which was devoted to a symposium on the drydocks at the Philadelphia Navy Yard. The guest of honor and principal speaker was Rear Admiral Frederic R. Harris, New York consultant and specialist in the field of drydock construction. Admiral Harris discussed the design of drydocks, covering the history of development in design from early models to the modern product now under construction at the Navy Yard. With the aid of slides he explained the general features of the Philadelphia docks, comparing them to previous installations such as the Pearl Harbor docks. Admiral Harris was followed by Dan Young, job engineer for Drydock Associates, who presented two reels of motion pictures. These films covered the construction of the Philadelphia project—from the transportation of Pennsylvania river mud to the flats across the river down to the last batch of tremie concrete. The colored pictures of the laboratory experiments in placing the tremie concrete helped all to a better understanding of the manner in which successive pours of concrete spread and form the finished product.

Providence Section

The Providence Section held a joint meeting with the Providence post of the Society of American Military Engineers, with Melvin M. Johnson, inventor of the Johnson semi-automatic rifle, as speaker. Mr. Johnson described the development of the design of this high-powered short-recoil shoulder arm to an audience of over a hundred. Before the meeting there was a dinner for the speaker.

St. Louis Section

The technical program at the March meeting consisted of a talk on the subject of "Air Raid Shelters"—given by John I. Parcel, St. Louis consultant. In his address Mr. Parcel emphasized the fact that the construction of air raid shelters involves problems quite different from those encountered in ordinary structural engineering. Protection from blast and splinter effect seems to be the desideratum, as it has been found impracticable to attempt to design shelters that will afford protection from direct hits. Mr. Parcel described the results of recent bomb tests at the Aberdeen Proving Ground and reported some of the tentative conclusions.

San Francisco Section

A meeting of the Junior Forum of the Section took place on March 26. Talks by two members of the Forum constituted the technical program. Arthur F. Liebscher, junior bridge engineer for the San Francisco–Oakland Bay Bridge, discussed the operation and maintenance of the structure. The other speaker was Dudley F. Stevens, editor of *Western Construction News*, whose subject was "The Engineer in the Publishing Business." The topic for general discussion was "Should the Government Set a Ceiling on Wages?" The discussion was opened by Norman Riffle and Edward Epstein.

Tacoma Section

At the March meeting of the Tacoma Section Dale L. Pitt, mining engineer and president of the Alabama-California Mines, discussed the subject of mining, describing his experiences as a mining engineer in foreign countries. Of particular interest was a description of the mineral resources of Australia, and the relation of these resources to the present war with Japan.

Texas Section

A two-day spring meeting of the Texas Section was held in Austin on March 13 and 14. The technical program got under way Friday morning with a talk by D. C. Greer, state highway engineer, who discussed some of the difficulties the Texas State Highway Department is facing in building roads without any new equipment and with the minimum use of equipment requiring rubber tires. In the last few months, however, over 400 miles of access roads have been put under construction. J. L. Dickinson, engineer of road design for the department, then outlined the modified designs calculated to meet the present shortage of vital materials. In connection with this shortage Fred Burkett, senior resident engineer for the department, described the construction

Group Attending Spring Meeting of Texas Section
Left to Right: J. T. L. McNew, E. B. Black, W. D. Dickinson, E. L. Myers, and R. F. Dawson

of a masonry arch bridge across the Brazos River, which utilized local materials. In the ensuing discussion it developed that the cost of this masonry is very reasonable, comparing favorably in price with reinforced concrete. J. A. Elliott, district engineer for the Public Roads Administration, emphasized the importance of highways in the defense effort, stating that raw materials and finished products are being delivered over our highways today in great quantity. A talk on trends in modern airport design—by L. C. Elliott, regional manager of the Civil Aeronautics Administration—concluded the Friday technical program. The presentation of certificates of life membership was a feature of the afternoon business meeting, and there was a dinner-dance in the evening.

On Saturday morning the annual breakfast for students and members was followed by the presentation of student papers in a competition for cash prizes presented by the Section. First prize of $25 went to Vann Allen, of the University of Texas; second prize of $15 to Paul O'Rourke, of Southern Methodist University; and third prize of $10 to Jim Pettigrew, of the Agricultural and Mechanical College of Texas. The remainder of the program consisted of a symposium on protection against aerial bombardment, the participants being Prof. Phil M. Ferguson, of the University of Texas, and Prof. A. A. Jakkula, of the Texas Agricultural and Mechanical College. Guests of the Section for the occasion included Society President E. B. Black, Directors Dickinson and McNew, and former Director E. R. Needles.

Tri-City Section

A talk on "The Problems of Power Supply" constituted the technical program at the March meeting of the Section, which was held in Moline on the 13th. This was given by H. A. Kleinman, electric production engineer for the Iowa-Illinois Gas and Electric Company, who explained the various means whereby the thermal efficiency of steam plants has been steadily increased during the past thirty years. His talk was illustrated with a series of charts that showed graphically the manner in which the losses have been reduced. George B. Massey, Director of the Society, was present and gave a short talk on Society affairs.

Utah Section

The March meeting of the Utah Section took the form of a joint session with the Utah Society of Professional Engineers. Following a dinner Dr. R. G. Frazier, one of the members of the most recent Byrd expedition to the South Pole, gave an illustrated lecture on the expedition. Dr. Frazier discussed the wild life found in the vicinity of "Little America" and described some of the hardships of living where the temperature is fifty or sixty degrees below zero.

At the April meeting Prof. A. Diefendorf, head of the civil engineering department at the University of Utah, spoke on "Engineering as a Factor in the So-Called Passive Resistance to Aerial Bombardment." Professor Diefendorf pointed out that many of the problems arising in connection with an aerial bombardment—problems of transportation, water supply, sewage, and public health—are primarily civil engineering in character and that the civil engineer should accordingly take the initiative in their solution. His talk was illustrated with motion pictures showing the results of bombing in England.

Wisconsin Section

A joint meeting of the Wisconsin Section and the Student Chapter at the University of Wisconsin took place on March 4. The technical program consisted of a talk on "New Developments in Sound Detection and Signal Devices," by Norman Blume, of the Wisconsin Telephone Company. Mr. Blume described the use of various waves, long and short-wave radio, infra-red rays, light, and sound, and discussed the possibilities in these fields with respect to military operation. He illustrated his talk with slides and demonstrated the operation of a cathode ray oscillograph.

Student Chapter Notes

College of the City of New York

The College of the City of New York Chapter reports that weekly meetings were held in March. Among the speakers heard on these occasions were Nathan I. Kass, sanitary engineer for the New York City Department of Public Works, who gave an illustrated lecture on sewage disposal plants in New York City; Shortridge Hardesty, New York consultant, who discussed the design and construction of the Rainbow Arch Bridge at Niagara Falls; and William Greene, of the American Bridge Company, who described the erection of the Passaic River Bridge.

Lehigh University

At a recent meeting of the Lehigh University Chapter W. J. Eney, associate professor of civil engineering at Lehigh, discussed his recent investigations in the field of model structural analysis. Professor Eney was assisted in his talk by Roger Kolm, a graduate student, and Robert Ashley, of the current senior class. Other meetings of the past few months have included talks on the Rainbow Bridge at Niagara Falls, the Unionmelt welding process, and the use of testing machines. Whenever possible, these talks are given by Lehigh engineering students or alumni—for instance, the next meeting will cover the research done by graduate students in the Fritz Engineering Laboratory.

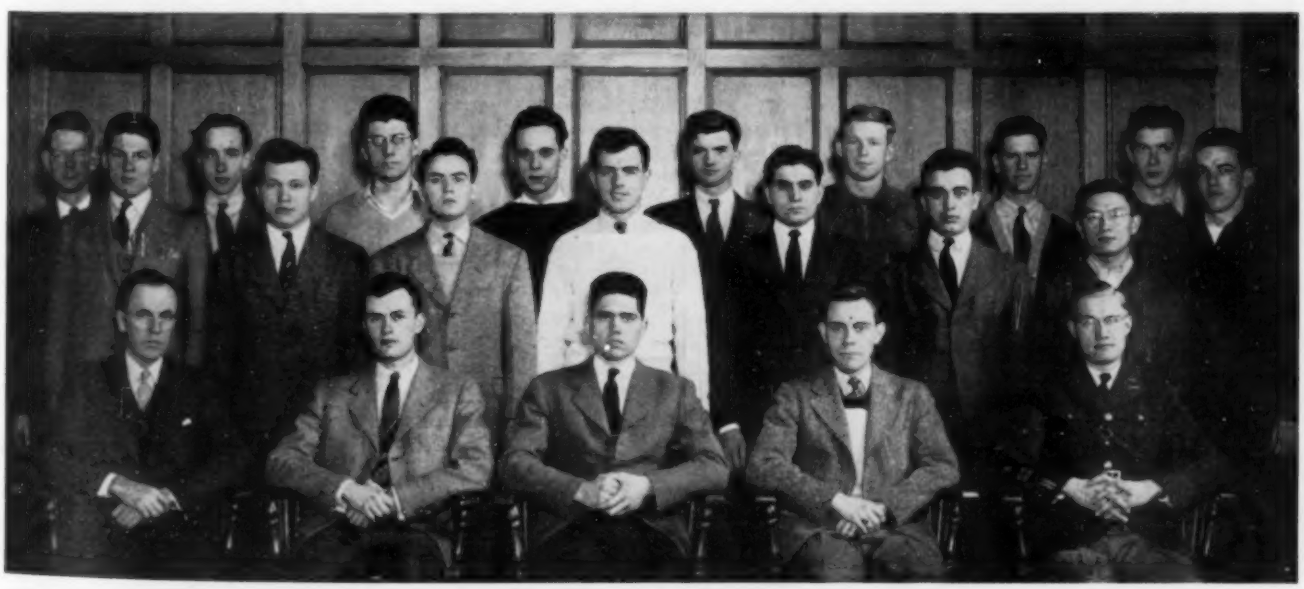
Syracuse University

The Syracuse University Chapter met on April 14 to perfect plans for the regional conference of Student Chapters for which the Syracuse Chapter is acting as host on April 25. The various committee heads gave their reports, and a tentative program was arranged. It was announced that the winner of the essay contest sponsored by the Chapter is Clyde Johnson, whose subject was "City Planning." The prize is $5.

Iowa State College

The Iowa State Chapter at Ames gets out a monthly two-page bulletin entitled "Civil-Chatter." Notices, contributions from faculty and students, personnel, and some lighter touches are included. For variety, the color of the bulletin changes from month to month. This sort of an effort means work for someone, but it is evidence of an active interest in promoting the Chapter. Aside from this labor, the cost of mimeographing both sides of a single sheet is insignificant in comparison with the return.

The Chapter reports that several interesting meetings have been held, the list of speakers including J. P. Lawlor, president of the General Filter Company, and Maurice Miller, president of the Iowa Section and the Iowa Engineering Society. Dean Emeritus Anson Marston spoke at the annual meeting on April 8.



Members of Lehigh University Student Chapter

ITEMS OF INTEREST

About Engineers and Engineering

Civil Engineering for June

Now, more than ever, drydocks are of national importance. The greatest development in this field in recent years involves the use of the tremie method, as described by Admiral F. R. Harris in the June issue. He traces the development of this type of construction through various stages at Erie Basin, New York, Honolulu, H.I., and now at two tremendous structures in Philadelphia. By this phenomenal development the largest types of drydock can be completed in two years, whereas formerly they took several times as long. This interesting paper originated in a meeting of the Philadelphia Section.

Recent developments in sanitation are dealt with by W. P. Cottingham in his article on "Sewage and Garbage Disposal at Gary, Ind." After giving a brief history of local development, he devotes the bulk of the paper to the performance of the new plant during a period of about a year. Of special interest are the results secured in treating strong acid pickling liquors from wire mills. His paper came from a session of the Sanitary Engineering Division at the Chicago Meeting last fall.

Hell Gate, at the western entrance to Long Island Sound, has long presented a problem to waterways engineers. As explained by Col. L. S. Dillon in his article, "Model Study of Tidal Currents in East River, New York," definite plans are under way to cure this critical situation. He details the hydraulic complications surrounding the problem, the solution that is being sought, the tests that have been performed at the U.S. Waterways Experiment Station, and the proposal for permanent improvement. These developments were first described by Colonel Dillon before the Waterways Division at its Annual Meeting in New York in January.

Also planned for the June number is a discussion of "Heavy Construction Equipment Used in War Work," by Kenneth F. Park. He dwells especially on the tremendous strides made in the handling of excavation, and includes a description of a gigantic unit consisting of tractor and conveying scraper, weighing together almost 100,000 lb empty, and 230,000 lb loaded. This machine and its smaller brothers are now making history on emergency construction work.

Welding Research Fellowship

A graduate fellowship in structural steel welding research available at Carnegie Institute of Technology offers an excellent opportunity to train for later positions in the welding industry. Applicants should hold a B.S. degree in civil engineering. The recipient of this fellowship will receive an annual stipend of $750, with opportunity of working for an M.S. degree, and freedom from tuition fees. He will be expected to devote one-half of his time, commencing September 1, 1942, to the Structural Welding Research program approved by the Structural Steel Welding Research Committee. Applicants should forward complete qualifications immediately to Professor F. M. McCullough, Head, Department of Civil Engineering, Carnegie Institute of Technology, Pittsburgh, Pa.

N. G. Neare's Column

Conducted by

R. Robinson Rowe, M. Am. Soc. C.E.

When our Engineers Club met in May, chit-chat at dinner proposed many solutions for that current politico-socio-ideological poser—the administration of impounded Japanese so as to unarm the disloyal and unharm the loyal. There was anticipation of the solution of Professor Neare's problem of the inter-racial population race, for several compared notes and one called out across the table, "You can take a rest tonite, Noah. Eight of us here agree on an answer—in 1219.7 years there will be more Japanese aliens than citizens in the United States, if births and deaths follow your schedule. By that time, incidentally, there will be 640 billion in each group, or one person for each 66 sq ft of the land—mountains, lakes, deserts, and all!"

"Cal Klater is usually right," spoke up one of our Westerners, "and I agree with his mathematics, but we in California are closer to the fine point of the problem in advanced multiplication—that Japanese aliens do not reproduce their class. Their children, the Nisei, are born citizens of the United States. So the answer is 'never!' "

"Even so," said Joe Kerr, "whether the Japanese majority is alien or citizen, something should be done, or our 200th President will be named Charlie Hirohitoson, or something. Instead of concentrating all of them at Camp Manzanar, let's transfer half of them to a new Camp Womanzanar. If the war lasts 100 years, our worries will be over."

"Better yet, take a tip from my state, New Mexico," put in another Westerner. "In the decade 1930–1939, our birth rate was 29.66 and our death rate 14.06. So in 1219.7 years, the descendants of our 531,000 New Mexicans will outnumber the Japanese 130 to 1!"

"This could go on and on," interrupted the Professor, "but there is barely time to state a new problem. Of course the Californian is wholly right, but I am inclined to commend you Easterners for your flawless multiplication and excuse you for ignorance of a law that heretofore has concerned you so little. Then, too, laws change.

"Our next problem concerns two cows tethered at the ends of a 100-ft rope that passes thru a hole in a straight fence between two pastures. If each cow grazes at the same rate, how much pasture can be grazed before one cow runs out of grass? Suppose one cow was replaced by a heifer with one-half the full bovine appetite, could more or less be grazed? How much difference would it make if the fence were replaced by a lone post?"

(*The gentleman from California was Life Member David E. Hughes; the eight Messrs. Klater were Benjamin Eisner, George T. Dean, Richard Jenney, Weston Gavett, Claude W. West, Byron A. Bledsoe, William C. Strasset, and Count Harvey. More precisely, the Klater-type solution yields a total population of 1,278,417,489,243 after 1,219.702562531800 years. Also acknowledged is a third full and precise solution of the February problem of the 12 S-C Air Bases, which came roundabout from William T. Moody, Captain, C.E.*)

Research on Sediment

A series of five reports dealing with various aspects of the measurement and analysis of sediment loads in streams has recently been issued. The reports cover the work carried on by several federal agencies having problems in the field, in cooperation with the Iowa Institute of Hydraulic Research. The federal agencies involved are the Corps of Engineers, the Geological Survey, the Bureau of Reclamation, the Office of Indian Affairs, the Flood Control Coordinating Committee of the Department of Agriculture, and the Tennessee Valley Authority.

Report No. 1, entitled "Field Practice and Equipment Used in Sampling Suspended Sediment," is a detailed review of the equipment and methods used in suspended sediment sampling from the earliest investigation to the present, with discussions of the advantages and disadvantages of the various methods and instruments. The requirements of a sampler that would meet all field conditions satisfactorily are set forth.

Report No. 2, "Equipment Used for Sampling Bed Load and Bed Material," deals with equipment for sampling bed load and bed material in a manner similar to that in which Report No. 1 covers suspended load.

Report No. 3, "Analytical Study of Methods of Sampling Suspended Sediment," covers an investigation of the accuracy of various methods of sampling suspended sediment in a vertical section of a stream, based on the latest developments in the application of turbulence theories to sediment transportation.

Report No. 4, "Methods of Analyzing Sediment Samples," describes and dis-

cusses many methods developed for determining the size of small particles in sediment analyses. Detailed instructions are given for many of the common methods in use for determining the particle size and the total concentration of sediment in samples as developed by agencies doing extensive work in these fields.

Report No. 5, "Laboratory Investigations of Suspended Sediment Samplers," describes experiments to investigate (1) the effect of various intake conditions on the accuracy of sediment samples and (2) the filling characteristics of slow-filling samplers under various conditions.

The publications range from 57 to 203 pages, all are illustrated, and all but Report No. 5 have bibliographies. Copies may be examined in the offices of the cooperating agencies. Because of the small number printed, distribution will be limited to those actively engaged in this field. Inquiries should be addressed to the District Engineer, U.S. Engineer Office, St. Paul, Minn. Reports 1 and 4 are $1 each, and Reports 2, 3, and 5 are 50 cents.

Ground Officers in the Army Air Corps

Efforts are being made to fill a large number of vacancies in the commissioned officer staff of the Army Air Corps for ground service. A variety of positions are open involving engineering of various kinds. Only those openings which call for civil engineering qualifications are noted in the following, listed under the various official code numbers.

Code No. 7. Construction Engineering Officers (Military experience not necessary)

Duties—Concerned with construction work in connection with buildings and services, with particular reference to air fields and bases. Must coordinate duties of other more specialized engineer officers.

Qualifications—Experience in construction of industrial building, public works, roads, heating, lighting, and other building services.

Code No. 28. Map Officer (Military experience not necessary)

Duties—To prepare and revise maps.

Qualifications—Should have at least 2 years of college training in an engineering or geological course. Working knowledge of cartography, lithography, photogrammetry and their application to the production and revision of maps would be helpful, including the transformation of necessary information from aerial pictures; also the establishment and application of photographic control required in map-making.

Code No. 30. Transportation Officer (Military experience desirable)

Duties—Responsible for transportation assigned to his organization, for proper maintenance of vehicles, for the training of drivers and for repair work. This includes only such minor repairs as may be effected by hand tools that normally come with any automotive vehicle. In emergencies, he is required to effect such necessary repairs with limited facilities in order to accomplish his mission, being responsible for the proper dispatch of motor transportation assigned to his organization.

Qualifications—He should be familiar with the operation of all types of motor transportation, from light vehicles to heavy-duty trucks, being able to instruct in the proper methods of driving all such types of vehicles. He should have a high sense of responsibility in the exercise of his duties.

Code No. 35. Camouflage Officers (Military experience not necessary)

Duties—To disguise or otherwise conceal all types of ground installations, military equipment, planes, hangars, buildings, air fields, and fuel dumps by means of camouflaged objects through visual observation and knowledge of camouflage methods.

Qualifications—Experience in any civilian occupation requiring a sense or knowledge of visual perspective, or a keen imagination.

Note—These officers are to be sent to the California Institute of Technology, Pasadena, Calif., as students for a three-months course in camouflage, instruction to start June 8, 1942.

In general, those who are classed as 1-A in the draft and are younger than 30 years are not eligible. Older men in the 1-A classification will have to secure temporary deferment to enable their applications to be considered. The general age limits will approximate the following:

2nd Lieutenants	—under 35 years
1st Lieutenants	—32–42 years
Captains	—36–60 years
Majors	—42–60 years

Under the classification as lieutenant, about 50–60% of the applicants might be included; with 30–35% as captains; and 5–10% as majors.

For those members who may be interested in this possibility of service, it is recommended that applications and information be secured from the regional Air Corps Offices most convenient to them. Arrangements for personal conference can then be made as required. It is understood, of course, that the location of possible service in the Air Corps is unrestricted.

Recent Ruling Affects Subcontracts on Defense Work

Engineers concerned with work being done by prime contractors for the Government, or by subcontractors for prime contractors, may already be familiar with the ruling of the Comptroller General under date of March 13. For those who may not be familiar with it the following summary has been prepared.

The ruling concerns subcontracts entered into by prime contractors on a cost-plus-a-percentage-of-cost basis, and holds that costs of work done by a subcontractor on this basis are not reimbursable to the prime contractor by the Government.

This ruling is based on the act of July 2 1940 (Public No. 703, 54 Stat. 712), which authorizes the Secretary of War to enter into various contracts in the interest of national defense. Section 1 provides ". . . that the cost-plus-a-percentage-of-cost system of contracting shall not be used under this section. . . ."

The specific case which recently opened this question was the request of Day and Zimmerman, Inc., working under a cost-plus-a-fixed-fee contract with the Government, for reimbursement for the sum of $10,280.84 paid to the Western Electro Mechanical Company for certain services rendered to Day and Zimmerman on the cost-plus-a-percentage-of-cost basis in fulfilling an ordnance subcontract.

The Chief of Ordnance contended that the subcontract on this basis was justified because of the pressing need for ordnance of all sorts, the specialized nature of the work, and the difficulty of obtaining subcontractors with the necessary experience to do the work required. In the endorsement of January 5, 1942, from the Chief of Ordnance to the Chief of Finance, it was further contended that Western Electro, and many other specialized manufacturers, refuse to set a fixed price on services which are susceptible to abnormal variation. Thus the prime contractor must agree to the subcontractor's terms or seek elsewhere in a "demand market," where few capable manufacturers, if any, are available. The alternative would be at least to sacrifice the objective of speed set for the defense program.

Further it was contended that the subcontractor can make a cost-plus-percentage-of-cost contract with the prime contractor because (1) he is dealing with an independent contractor and not directly with the Government; (2) he has been held not to be in privity with the Government for the purpose of making a claim on it; and (3) therefore he is not restricted by the inhibition covering the Government's contractual relations with the prime contractor.

These views of the Ordnance Department were reviewed by the Comptroller General. In his decision of March 13, previously referred to, he states:

"While careful consideration has been given to the views of the Ordnance Department with respect to the legality of subcontracts entered into by a prime contractor on a cost-plus-a-percentage-of-cost basis, I am constrained to hold that such type of subcontracts are in contravention of the spirit and purpose of the act of July 2, 1940, providing that 'the cost-plus-a-percentage-of-cost system of contracting shall not be used under this section.' It is evident that the prohibition against this form of contracting could be substantially evaded and the purposes thereof defeated were it not applied to the performance of that part of the contract work sublet by the prime contractor to others. In apparent recognition of this possibility, regulations have been issued by your Department wherein the officials thereof have been instructed to refuse 'to give their approval to subcontracts proposed to be entered into upon a cost-plus-a-percentage-of-cost basis.' See Supplement No. 6

to the Manual for the Construction Division, Book IV, Part II, dated Oct. 15, 1941, and Construction Division letters Nos. 340, 350, and 478."

On this basis the Comptroller General disallowed the request of Day and Zimmerman for reimbursement.

Recent Changes in Federal Works Agency

In line with the present policy of streamlining the Federal Works Agency into a centralized and unified national public works agency, General Order No. 67, dated March 4, 1942, abolished the Defense Public Works Administration by transferring its functions to the new Federal Works Agency, and established the office of Chief Engineer of the Federal Works Agency. The man chosen for this important position is Col. William N. Carey, M. Am. Soc. C.E., Director of the Society for District 10. Colonel Carey was consulting engineer of St. Paul, Minn., until recently, when he was called to active duty in the U.S. Engineer Corps and stationed at Jacksonville, Fla.

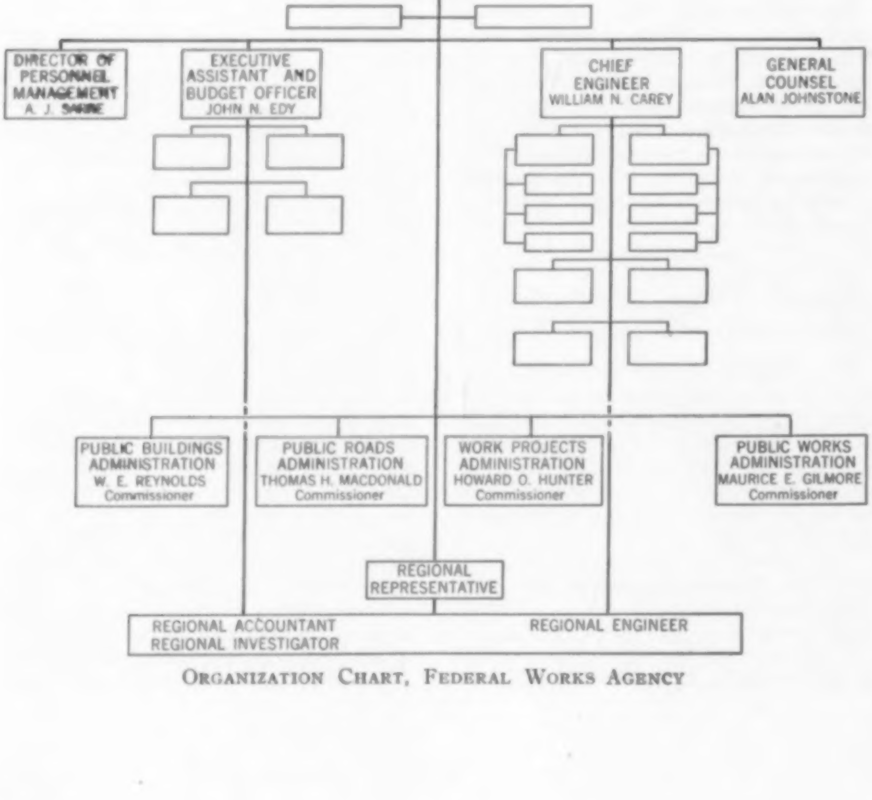
Under this new centralized FWA set-up the Chief Engineer, General Counsel, Budget Officer, and Director of Personnel all report directly to the Deputy Administrator, who in turn reports to the Administrator. The Administrator is Brig. Gen. Philip B. Fleming, and Baird Snyder, III, is the Deputy Administrator. A. J. Sarré is Director of Personnel Management; John N. Edy, M. Am. Soc. C.E., is Executive Assistant and Budget Officer; Col. William N. Carey is Chief Engineer, and Alan Johnstone is General Counsel.

This centralized organization is prepared to carry on any public works construction regardless of its nature, and thus it takes over the functions of the old PWA and the old Defense Public Works Administration. All housing projects have been removed from the Federal Works Agency and placed in a new National Housing Agency under John B. Blandford, Jr., Administrator.

The policy of the old Public Works Administration as a joint federal-municipal organization will not carry over in this new public works set-up of FWA. The new PWA will be entirely concerned with construction and, if possible, have nothing to do with management. Even where there is need for federal construction work in connection with housing, Army, Navy, or Defense Plant Corporation projects, the new PWA, as a component part of the FWA, will do the construction for the Federal Government but will not take a hand in management. This new plan also removes from the new PWA the financing function of the old PWA. The new PWA is nothing more or less than the construction division of the FWA. To accomplish this and to expedite the defense public works program, the old Defense Public Works Administration is being wiped out entirely and its organization is being set up around the framework of the new PWA, except that it now functions, not as a separate entity, but as an integral part of this new Federal Works Agency. This makes the new FWA a simple straightforward engineering and construction organization with as little concern as possible given to social or management problems.

With the old PWA revamped, the Defense Public Works Administration abolished, and all housing projects put under a new agency, the FWA is left with four separate branches—Public Buildings Administration, Public Roads Administration, Work Projects Administration, and Public Works Administration. These four remaining administrations, as now set up, are put under a centralized control where formerly each administration was a separate entity functioning entirely on its own. Each of the four divisions of the FWA is under a commissioner; W. E. Reynolds, M. Am. Soc. C.E., for the Public Buildings Administration; Thomas H. MacDonald for the Public Roads Administration; Howard O. Hunter for the Work Projects Administration; and Maurice E. Gilmore, M. Am. Soc. C.E., for the Public Works Administration.

The field offices of the old PWA now become regional offices of the FWA. There are seven throughout the United States.



Organization Chart, Federal Works Agency

Accelerated Engineering Courses

Results of an inquiry recently instituted by the Society for the Promotion of Engineering Education indicate a general adherence to the idea of concentrating courses for engineering students. A total of 103 out of 126 colleges reporting are adopting the plan. Instead of shortening the courses, the speeding up is obtained by minimizing the vacation period so that in most instances the college year will consist of 48 weeks in every 12 months.

Methods of accomplishing this schedule vary. Most popular is the change to three equal terms as adopted by 36 colleges. About the same number, 34, will simply add summer sessions; while 16 will have four terms of equal length. The speed with which the plans are being inaugurated is indicated by the fact that in two-thirds of the colleges every undergraduate engineering student will be following an accelerated schedule.

Dates for graduation are advanced as follows: May, 17 colleges; first half of June, 53 colleges; second half of June, 23 colleges; and July, 4 colleges. Most of the colleges—77%—will enroll freshman classes during the summer. Registrations are divided into three equal groups comprising two, three, and a combination of one and four registrations per year. The cost of additional college work will be at the same rates as at present in over 80% of the institutions; a small number will increase their rates; and about 15% will lower them.

These interesting data have been collected by a committee of the S.P.E.E. under D. B. Prentice, president of Rose Polytechnic Institute.

Non-Defense Construction Rigidly Curtailed

On April 9, the War Production Board called a halt to non-essential construction. Effective immediately, Conservation Order L-41 prohibits the start of unauthorized construction projects which use material and construction equipment needed in the war effort. It also places all new publicly

and privately financed construction under rigid control, except for certain strictly limited categories.

The action was taken by the War Production Board, Division of Industry Operations, because the war requirements of the United States have created a shortage of materials for war production and construction. It is in the national interest that all construction which is not essential, directly or indirectly, to the successful prosecution of the war, and which involves the use of labor, material, or equipment urgently needed in the war effort, be deferred for the duration of the emergency.

The order does not apply to maintenance and repair work. Nor does it apply to construction already under way. It applies to new construction, defined as the initiating of "construction by physically incorporating into any construction material which is an integral part of the construction."

The order does not apply to new residential construction in urban or rural areas where the cost of the project during any twelve-month period is $500 or less, or to farm building construction where the cost is $1,000 or less, or to commercial industrial, recreational, institutional, highway, roadway, subsurface and utilities construction, whether publicly or privately financed, where the cost is $5,000 or less.

Construction other than that mentioned cannot go forward, nor can dealers supply material for it, without authorization. Such authorization must be evidenced either by the buyer's having a priority rating or certificate (not including PD-1 or PD-1A certificates) or having a special authorization. The latter may be obtained by showing the essential nature of the proposed work through filing forms PD-200 or PD-200A with the local office of the Federal Housing Administration.

It will be noted that this order does not "freeze" dealer inventories in the ordinary sense of those words.

Conservation Order No. L-41, which gives the new regulations in detail, may be secured from the War Production Board, Division of Industry Operations, or from the Chamber of Commerce of the United States, Washington, D.C.

Federal Housing Has New Unified Agency

By Executive order dated February 24, 1942, President Roosevelt consolidated the various housing activities of the Government into one National Housing Agency under John B. Blandford, Jr., Administrator. The Administrator will perform the housing functions formerly vested in the Federal Loan Administrator, the Federal Works Administrator, and the Coordinator of Defense Housing. The new agency consists of three principal constituent units—the Federal Home Loan Bank Administration, the Federal Housing Administration, and the Federal Public Housing Authority.

The Federal Home Loan Bank Administration is under the direction of John Fahey, Commissioner, and consolidates the financing, home-ownership, and construction functions formerly vested in the Federal Home Loan Bank Board. Federal Home Loan Bank System, Federal Savings and Loan Insurance Corporation, Home Owners' Loan Corporation, and the U.S. Housing Corporation (for liquidation).

The Federal Housing Administration is under the direction of Abner Ferguson, Commissioner, and exercises all the functions formerly vested in the Federal Housing Administration when it was a part of the FWA. It will continue to guarantee or insure mortgages placed on homes by banks and lending institutions.

The Federal Public Housing Authority is under the direction of Leon Keyserling, Acting Commissioner, and is an amalgamation of the agencies and personnel engaged in constructing housing with public money. This Authority carries on the functions formerly vested in the U.S. Housing Authority, Defense Homes Corporation, Non-Farm Public Housing (from Farm Security Administration), and Defense Public Housing (except on Army and Navy reservations). Such Defense Public Housing, except on Army and Navy reservations, has heretofore been divided among the Federal Works Agency, the U.S. Housing Authority, the Public Buildings Administration, the Division of Defense Housing, the Mutual Ownership Defense Housing Division, the War Department, the Navy Department, and the Farm Security Administration. The housing now owned by the United States and located on military or naval reservations, posts, or bases was transferred to the jurisdiction of the War or Navy Department.

Construction Materials Restricted

New limitations are being imposed on critical construction materials. Announcement to this effect governing a defense housing critical list has become effective February 24 by order of the War Production Board, Division Industry Operations, Housing Priorities Branch. This list gives materials and accessories whose use is permitted. Steel for reinforcing and other uses, structural materials, hardware, electrical and sanitary equipment, and other provisions are covered in detail.

Instructions regarding critical materials used on highways, bridges, and culverts were promulgated on March 18. A first list covers especially critical materials including rubber, burlap, and a number of metals such as alloys, steel, aluminum, copper, lead, and others. A second grouping includes cork, cotton duck, carbon steel, and other materials that are somewhat less critical. Finally there is a third list, containing asphalt, cement, concrete, glass, limestone, marble, wood, and like materials which are more available and hence useful for substitutes. Those with specific problems in this field will wish to secure the necessary information from the Public Roads Administration in Washington, as given in its general administrative memorandum No. 158 dated March 24, 1942.

Brief Notes

Civil service registers are about to be established from which vacancies in the Arizona State Department of Health may be filled as they occur from time to time, including State Director of Sanitary Engineer, salary $300–$350; and sanitary engineer, $200–$250. Applications are expected to close about the middle of June. Blanks and all necessary information may be secured by writing the Merit System Council, 308 Home Builders Building, Phoenix, Ariz.

* * * *

The Department of Public Health at the Massachusetts Institute of Technology is offering an accelerated program of public health training beginning June 8, and allowing for the completion of a master's degree on February 6. These training programs are organized for public health engineers, health educators, and public health laboratorians, as well as for administrators. Special summer courses are also being offered.

* * * *

Announcement has been made of the spring meeting of the New York State Sewage Works Association—to be held at the Hotel Ten Eyck in Albany on June 5 and 6.

* * * *

A short summer course in city and regional planning will be held at the Massachusetts Institute of Technology during the three weeks beginning July 13. The program, sponsored jointly by the Institute and the American Society of Planning Officials, will be divided into four sections: City and Regional Planning, Planning Legislation, Planning Administration, and Techniques of Planning. Emphasis will be placed on new demands made on the planning profession during the war and post-war periods. Opportunity will be given for the study of design and research problems under supervision. The fee for the entire program is $55, and less for separate sections. For further information, and for enrolment in the program, address Prof. F. J. Adams, Division of City Planning, M.I.T., Cambridge, Mass. Applications should be sent not later than July 6.

NEWS OF ENGINEERS

Personal Items About Society Members

Harold M. Lewis, for a number of years chief engineer and secretary of the New York Regional Plan Association, Inc., announces the opening of a consulting office at 18 East 48th Street, New York. He will specialize in city and regional planning and zoning and will report on special problems.

C. E. Myers, colonel, Corps of Engineers, U.S. Army, is now stationed at Chambersburg, Pa., in the capacity of area engineer. Until recently he was at Fort Belvoir, Va.

R. W. Remp has been promoted from the position of general superintendent for the Dravo Corporation at Fishkill, N.Y.,

to that of vice-president, with headquarters in Wilmington, Del.

Gerald E. Arnold, for the past eleven years chief water purification engineer for the San Francisco Water Department, is now regional sanitary engineer for the U.S. Public Health Service. Mr. Arnold, who has a major's commission, will be in charge of civilian sanitation for the Ninth Corps Area. He will be succeeded as acting chief water purification engineer by H. C. Medbery, formerly water purification engineer.

Lorin T. Blodget is now cost engineer for the United Engineers and Constructors, Inc., with headquarters at Alabama City, Ala.

Andrew J. Rominiecki, lieutenant, Corps of Engineers, U.S. Army, has been assigned to Fort Lewis, Wash.

Clarence P. Schantz was recently appointed assistant chief engineer of maintenance of way for the Pennsylvania Railroad at Philadelphia, Pa. He was formerly assistant engineer of bridges.

Frederick H. Richardson, lieutenant colonel, Corps of Engineers, U.S. Army, has been transferred from Camp Dix, N.J., where he was Constructing Quartermaster, to the District Engineer's Office at Philadelphia, Pa.

Jeremiah Tempone has returned from Puerto Rico, where he served as construction engineer for the Public Buildings Administration of the Federal Works Agency, and is engaged on the new housing project at Chester, Pa.

R. F. McCune is now chief of the construction division for the Bluebonnet Constructors, who have a contract for the Bluebonnet Ordnance Plant now being built at McGregor, Tex. Mr. McCune's headquarters are at McGregor.

Wendell P. Brown, retired Cleveland consultant, has been awarded a citation by the Ohio Society of Professional Engineers for "meritorious achievement in the promotion of the professional welfare of engineers." This honor, which is believed to be the first of its kind to be given by the Ohio Society of Professional Engineers, was bestowed at the sixty-third annual convention of the organization in February. Former president and chief engineer of Wendell P. Brown and Associates, Mr. Brown is past-president and honorary member of the Cleveland Engineering Society.

Edward N. Todd, assistant engineer for the U.S. Engineer Office, has been transferred from the Jeffersonville (Ind.) Quartermaster Depot to the Nashville (Tenn.) District, where he is employed on the construction of the Smyrna Air Base.

Ralph Reed has resigned as city engineer of Watertown, S.Dak., in order to accept a position in a civilian capacity with the Corps of Engineers at Fort Peck, Mont.

Carroll L. Mann, Jr., captain, Corps of Engineers, U.S. Army, has been appointed acting area engineer in charge of all construction at Fort Bragg, Fayetteville, N.C.

James H. Turner, until lately assistant engineer for the city of San Francisco, has been appointed manager and chief engineer of the Hetch Hetchy water supply for the San Francisco Public Utilities Commission.

Michael A. Fouhy was recently transferred from the Boston Navy Yard, where he has been associate civil engineer, to the U.S. Navy Air Station at Brooklyn, N.Y., where he will serve in the capacity of civil engineer and chief draftsman.

Gordon H. Butler, lieutenant colonel, Corps of Engineers, U.S. Army, has been named area engineer in the District Engineer Office at St. Louis, Mo.

E. W. Bauman, formerly highway materials engineer for the Republic Steel Corporation at Cleveland, Ohio, has been named technical adviser to the nonmetallics mining branch of the War Production Board, stationed in Washington, D.C.

Ellis L. Armstrong, who is on the staff of the U.S. Bureau of Reclamation, has been transferred from the Deer Creek Dam at Provo, Utah, on which he was assistant engineer, to serve in a similar capacity on the Anderson Ranch Dam in Idaho.

Irvin L. Simmons has retired from the post of bridge engineer for the Chicago, Rock Island and Pacific Railway after thirty-nine years in the engineering department. Mr. Simmons had been bridge engineer since 1909.

Frank W. Edwards, until lately senior hydraulic engineer for the Panama Canal, has accepted a position as senior engineer in the South Atlantic Division of the U.S. Engineer Office, with headquarters at Atlanta, Ga.

Richard Irvin, architect and engineer of Pittsburgh, Pa., has been appointed WPA director for Pennsylvania.

Edward W. Stearns is now project engineer for the Corbetta Construction Company on warehouse construction at Columbus, Ohio.

Leon Maslan, formerly assistant civil engineer for the U.S. Soil Conservation Service, at Pocatello, Idaho, has accepted a position in a similar capacity in the U.S. Engineer Office at Savannah, Ga.

Theodore R. Kendall has resigned as engineering editor of the *American City Magazine* in order to devote his full time to the editorship of *Contractors and Engineers Monthly*. However, he will act as consulting editor for the former publication. He will be succeeded as engineering editor by William S. Foster, formerly with the Stanley Engineering Company, of Muscatine, Iowa.

Glenn B. Woodruff in now chief engineer for Utah-Pomeroy-Morrison at Provo, Utah. Until recently he maintained a consulting practice in Berkeley, Calif.

Earl I. Brown, colonel, Corps of Engineers, U.S. Army, has returned to the army's retired list after holding the post of District Engineer at Wilmington, N.C., since 1940.

Elmer W. Gain, who is on the staff of the U.S. Soil Conservation Service, has been transferred from Dayton, Ohio, to Fowler, Ind. His position will be that of engineer assisting the soil conservation districts in the northwestern part of the state.

Dana Young recently resigned as head of the civil engineering department at the University of Connecticut in order to become professor of applied mechanics at the University of Texas.

M. E. Salsbury, senior assistant chief engineer for the Los Angeles County Flood Control District, has been appointed acting chief engineer for the District, replacing Harold E. Hedger who was recently called to active duty in the Corps of Engineers. Mr. Hedger has the rank of captain and is located in Washington, D.C.

William H. Hall has resigned as assistant city engineer of Oklahoma City, Okla., in order to join the teaching staff of the Oklahoma Agricultural and Mechanical College.

Edward M. Killough, valuation engineer for the West Maryland Railway and president of the Maryland Section, has been called to active duty in the U.S. Army with the rank of captain. He is stationed at Camp Lee, Va.

George R. Schneider, since 1937 head of the design section for the U.S. Engineer Department at Little Rock, Ark., has been appointed chief of the engineering division.

Gibb Gilchrist has been granted a brief leave of absence from his duties as dean of engineering at the Agricultural and Mechanical College of Texas in order to assist the War Production Board in Washington, D.C., on highway priority matters. Dean Gilchrist was formerly Texas State highway engineer.

Ray Adams is now with the Coast Artillery Corps with headquarters in Washington, D.C. He has a major's commission. He was previously senior engineer in the Bureau of Bridges of the Missouri State Highway Commission.

Charles Nordhaus Goldenberg has been called to active duty with the rank of lieutenant commander. He is stationed at Chickasaw, Ala.

DECEASED

Francis Webster Blackford (M. '88) civil and mining engineer of Los Angeles, Calif., died in that city on March 25, 1942, at the age of 82. Mr. Blackford had assisted in building the Oregon Short Line Railroad and was chief engineer during construction of the Cerro de Pasco Railway over the Andes in Peru. He had also been engaged in railway building in Mexico City and, at one time, maintained a consulting practice at Columbus, Ohio.

Edwin Ford Dawson (M. '08) retired engineer of Philadelphia, Pa., died at his home in that city on February 25, 1942. Mr. Dawson, who was 82, was for many

years resident engineer in Philadelphia for the Reading Company.

Manuel Victor Domenech (M. '06) treasurer for the government of Puerto Rico, died at San Juan, Puerto Rico, on March 15, 1942, at the age of 72. Mr. Domenech occupied a conspicuous place in public life for over forty years, having served as treasurer, financial adviser to several governors, and as acting governor. During President Wilson's administration he was Commissioner of the Interior, the first Puerto Rican so appointed. Educated at Lehigh University, Mr. Domenech was a staunch friend of the United States.

Arthur Sylvester Douglass (M. '32) construction engineer for the Detroit Edison Company since 1920, died in Detroit on March 6, 1942. He was 59. Mr. Douglass was active in the work of the Detroit Building Congress and a former member of the Detroit City Plan Commission. During the war he served as a lieutenant colonel of ordnance, receiving a special letter of commendation for his services as chief of the Boston Ordnance District.

Alex Dow (M. '06) chairman of the board of the Detroit Edison Company and Honorary Member of the Society, died at Ann Arbor, Mich., on March 22, 1942. On April 12 Mr. Dow would have been 80. The recipient of many engineering honors, he was elected an Honorary Member of the Society in 1936. A photograph and more detailed biographical sketch appear in the Society Affairs section of this issue.

Carl E. Downing (Assoc. M. '15) of Oxford, Miss., died there recently. Mr. Downing, who was 56, was for twelve years district engineer for the Mississippi State Highway Department. Earlier in his career he was engineer for the Belzoni (Miss.) Drainage District, and at one time was city engineer of Belzoni.

Henry Newton Francis (M. '88) died at Cranston, R.I., on February 28, 1942. Mr. Francis, who was in his eighty-ninth year, was one of the Society's oldest members, having become a Junior in 1876 and a Member in 1888.

Arthur Sanford Gelston (M. '35) assistant district engineer for the Shipbuilding Division of the Bethlehem Steel Company at San Francisco, Calif., died on March 3, 1942. He was 54. Mr. Gelston's earlier career included experience with the Sacramento Valley Irrigation Company, the California Development Company, and L'Hommedieu and Wilson, Inc. From 1930 until recently he was resident engineer for the California Joint State Highway District at Oakland, Calif.

George William Hawley (M. '29) deputy state engineer in charge of dams for the Division of Water Resources of the California Department of Public Works, died at Sacramento, Calif., on March 17, 1942. He was 52. Before joining the Department of Public Works Mr. Hawley had been connected with the Oregon Electrical Company, the San Joaquin Irrigation District, and the East Bay Water Company in California.

Virgil Henry Hewes (M. '04) who retired in 1938 after twelve years with the Federal Light and Traction Company, New York, N.Y., died in that city on March 21, 1942. Mr. Hewes, who was 83, was at one time manager and treasurer of the Zwoyer Fuel Company, of New York, and had also been associate engineer for J. A. L. Waddell of the same city. During the war he served as a government engineering expert at Bridgeport, Conn.

Francis Thompson Hillman (M. '25) since 1924 construction engineer for Winston Brothers, of Minneapolis, Minn., died suddenly at Texarkana, Tex., on February 23, 1942. He was 59. Mr. Hillman had been at Texarkana for the past few months engaged on the construction (for Winston Brothers) of the Lone Star Ordnance Plant. The projects on which he was employed while with Winston Brothers include Diablo Dam, Cle Elum Dam, Morris Dam, and the East Iron Mountain and Coxcomb Tunnels of the Colorado River Aqueduct. More recently he assisted in reorganizing and constructing Brea Dam (a part of the Southern California flood-control plan).

Minoru Inaba (Assoc. M. '40) designing engineer for Parsons, Klapp, Brinckerhoff and Douglas, of New York, N.Y., died on August 25, 1941, though the Society has just heard of his death. He was 34. A native of Japan, Mr. Inaba was educated in this country and spent his professional career here. From 1929 to 1937 he was structural steel draftsman for the Bethlehem Steel Company, and from the latter year on structural steel designer for Parsons, Klapp, Brinckerhoff and Douglas.

Charles Melville Jenkins (Assoc. M. '16) died at Sitka, Alaska, on March 12, 1942. Before going to Alaska two years ago as structural inspector for Siems Drake Puget Sound, Colonel Jenkins was for a number of years sales engineer for the E. J. Bartells Company, of Seattle, Wash. A lieutenant colonel in the Reserve Corps of the U.S. Army, he had recently been ordered to active duty at Seattle but died on the eve of sailing. During the World War he served overseas with the 20th Engineers.

Frank Caleb Kelton (M. '26) professor of civil engineering at the University of Arizona, died recently at the age of 60. Professor Kelton went to the University of Arizona in 1907—first as assistant irrigation engineer at the Agricultural Experiment Station there. He served as assistant professor of civil engineering from 1912 to 1918, and associate professor in 1918 and 1919. From the latter year on he was head of the department and full professor.

The Society welcomes additional biographical material to supplement these brief notes and to be available for use in the official memoirs for "Transactions."

Raymond Robb Marsden (M. '26) engineer for the Atlas Powder Company, of Wilmington, Del., died in that city on March 11, 1942. He was 57. From 1919 to 1925 he was professor of civil engineering at his alma mater, the Thayer School of Civil Engineering at Dartmouth College, and from 1925 to 1933 dean of the school. Before going to the Atlas Powder Company in 1940 he had served as engineer for the Public Works Administration in New Hampshire and Vermont and, at one time, was engineer for the Vermont State Highway Department.

John Glenn Mason (M. '30) since 1926 Nebraska state bridge engineer, Lincoln, Nebr., died on March 26, 1942, at the age of 59. At one time Mr. Mason had maintained a private consulting practice, and he was also (1920 to 1923) on the civil engineering faculty of the University of Nebraska. During the war he served in the Corps of Engineers, U.S. Army, with the rank of lieutenant.

Fred Morley (M. '96) retired engineer and educator, died at Ann Arbor, Mich., on March 27, 1942. Mr. Morley, who was 86, was an instructor in civil engineering at the University of Michigan from 1889 to 1894, and professor of civil engineering at Purdue University from 1894 to 1899. From the latter year to 1902 he was resident engineer for the U.S. Engineer Office on the St. Clair Flats (Michigan) Survey, and from 1902 to 1905 was on the Illinois River Survey. He retired in the latter year.

John Francis Murray (M. '06) retired engineer, died at his home at Moylan, Pa., on March 16, 1942. He would have been 70 in a few weeks. Until his retirement in 1930 Mr. Murray was for many years assistant to the chief engineer of the Pennsylvania Railroad. He served in the Spanish-American War with the first regiment of volunteer engineers. In 1940 the Engineers' Club of Philadelphia elected him to life membership.

George Coffin Power (M. '93) retired engineer of Ventura, Calif., died on February 10, 1942, at the age of 90. Mr. Power's death closed an engineering career of almost sixty years in Ventura County—he constructed the first Ventura city sewer system, made preliminary surveys for the city of Oxnard, and served on the Ventura City Plan Commission and the Ventura City Council. Later he served as administrator of several great California estates and, at the time of his death, was owner of the famous Power lemon ranch near Ventura.

Fred Elmer Rightor (M. '18) secretary of the Texas State Board of Registration for Professional Engineers, Austin, Tex., died suddenly on March 1, 1942. He was 59. Mr. Rightor was with the Illinois Central Railroad from 1900 to 1906, and from 1908 to 1938 was with the Warren Brothers' bitulithic interests in Texas. Coincidentally for part of this period (1919 to 1939) he was president and treasurer of the Southwest Bitulithic Company. He had been secretary of the Texas State Board of Registration for Professional Engineers since its inception in October 1937.

FRANKLIN AUGUSTUS SNOW (M. '05) civil engineer of Cambridge, Mass., died there on March 19, 1942. Mr. Snow, who was 87, supervised the building of railroads in several South and Central American countries, and since 1901 had been contractor for underground conduit systems for the electric illuminating companies in Boston and other New England cities.

ARTHUR NEWELL TALBOT (M '88) professor emeritus of engineering at the University of Illinois, died in Chicago, Ill., on April 3, 1942. Dr. Talbot, who was 84, was on the engineering staff at the University of Illinois from 1885 until his retirement in 1926. He had served as Director, Vice-President, and President of the Society, and in 1925 was elected Honorary Member. A photograph and more detailed biography appear elsewhere in this issue.

HARRY TUCKER (M. '24) associate member of the North Carolina State Utilities Commission, died at Raleigh, N.C., on March 18, 1942, at the age of 52. Since early in 1941 Professor Tucker had been on leave of absence from North Carolina State College, where he was professor of highway engineering and director of the Engineering Experiment Station. He was the author of many engineering textbooks, and during the World War served in the Corps of Engineers, with the rank of captain.

DANIEL LAWRENCE TURNER (M. '05) retired New York engineer, died at Norfolk, Va., on March 12, 1942. He was 72. From 1900 until his retirement in 1929 Mr. Turner was associated with the planning and development of rapid-transit lines in New York, having held executive posts with the Rapid Transit Construction Commission, the Public Service Commission, the Rapid Transit Commission, and the Board of Transportation. He planned the joint vehicular and rapid-transit highway in Detroit and served as consulting engineer on rapid transit there for many years. Mr. Turner also served as consultant for the Pittsburgh and North Jersey Transit Commissions.

CARL ROBERT WEIDNER (Assoc. M. '10) chief engineer for the Sinclair Refining Company, of Independence, Kans., died in August 1941, though word of his death has just reached the Society. He was 58. Mr. Weidner was instructor in engineering at the University of Wisconsin from 1909 to 1916; chief engineer for the Prairie Pipe Line Company from 1916 to 1932; and chief engineer for the Sinclair Prairie Pipe Line Company from 1932 to 1936. In the latter year he became connected with the Sinclair Refining Company.

THOMAS EDWARD WIGGINS (Assoc. M. '35) since 1935 field engineer for the Federal Works Agency of the Works Progress Administration, Oklahoma City, Okla., died recently. Mr. Wiggins, who was 47, had been connected with a number of organizations, including the California State Highway Department, the Portland Cement Association, and the Southwestern Dredging Corporation. He had acted as consultant for various municipalities in Oklahoma and, during the World War, served in the U.S. Corps of Engineers.

Changes in Membership Grades

Additions, Transfers, Reinstatements, and Resignations

From March 10 to April 9, 1942, Inclusive

ADDITIONS TO MEMBERSHIP

ABRINA, RIZAL DONALDO (Jun. '42), Junior Engr., U.S. Engr. Office, Wright Field, Dayton, Ohio.

ALLEN, OCIE CARL (Jun. '42), Care, Pan Am Airways, Inc., First National Bank Bldg. Brownsville, Tex.

BARRY, FRANCIS THOMAS (Jun. '41), Field Engr., Crucible Steel Co. of America, Fourth St., Harrison, N.J. (Res., 31–22 Thirty-Fourth St., Long Island City, N.Y.)

BENNETT, JOSEPH HENRY (Jun. '41), Eggers, Colo.

BINGHAM, JAY RULON (Jun. '41), Junior Engr., Dept. of Interior, U.S. Bureau of Reclamation, Knight Block, Provo, Utah.

BIXBY, PAUL (Jun. '41), Civ. Engr., Special Eng. Div., The Panama Canal, Box 317, Cocoli, Canal Zone.

BONA, LOUIS EUGENE (Jun. '42), Asst. Civ. Engr., U.S. Engrs., 3d and Broadway (Res., 115 North Cedar), Little Rock, Ark.

BOWLES, CHARLES ALFRED (Assoc. M. '42), Res. Engr., State Highway Dept., Box 191, Oneonta, Ala.

BRIGHTBILL, LINWOOD JAMES (Assoc. M. '42), Structural Designer, Siems Drake Puget Sound, 2929 Sixteenth, S.W. (Res., 1715 Sunset Ave.), Seattle, Wash.

BROWN, MAURICE WESLEY (Jun. '41), With Colorado Fuel & Iron Corp. (Res., City Y.M.C.A.), Pueblo, Colo.

BURKETT, FRED (M. '42), Senior Res. Engr., State Highway Dept., 2309 Montgomery, Fort Worth, Tex.

BURNS, ROBERT CHESTER (Jun. '42), Junior Structural Engr., TVA, 200 Arnstein Bldg. (Res., 704 Sixteenth St.), Knoxville, Tenn.

CABELL, ROY EDWARD (Assoc. M. '42), Asst. Engr., U.S. Geological Survey, Holbrook, Ariz.

CAMELLERIE, JOSEPH (Jun. '41), Junior Naval Archt., Navy Yard (Res., 4833 Walnut St.), Philadelphia, Pa.

CHONG, WAH JAN (Jun. '41), Junior Engr., U.S. Engr. Office (Res., 1518 Thurston Ave.), Honolulu, Hawaii.

CHRISTIANSEN, HAROLD SANFORD (Jun. '41), Ensign, U.S.N., Puget Sound Navy Yard, Bremerton, Wash.

CLAIRE, WILLIAM HAYWARD (Assoc. M. '42), Lt., U.S. Army, Officers Training School, Quartermaster Replacement Training Center, Fort Warren, Wyo.

COOK, LAWRENCE HARVEY (Assoc. M. '42), Pres., Cook Research Laboratories, Ltd., 950 Crane St., Menlo Park, Calif.

COON, THOMAS ROBERT (Jun. '41), Design Engr., State Highway Dept., Lansing (Res., 615 Lake Ave., Hancock), Mich.

COX, EVERETT LEE (Jun. '41), Asst. Office Engr., J. A. Terteling & Sons (Res., 131 East Maple), Walla Walla, Wash.

COX, ROSS EARL (Jun. '41), Structural Engr., El Paso Natural Gas Co., Bassett Tower, 12th Floor, El Paso, Tex.

DALTON, JAMES CECIL (Jun. '41), Dist. Materials Technologist, State Bureau of Highways, Box 152 (Res., 1200 Sherman Ave.), Coeur d'Alene, Idaho.

DAVIS, HILTON KUNZE (Jun. '41), Junior Engr., Corps. of Engrs., War Dept., Utilities Dept., Camp Wheeler, Ga. (Res., 3514 South Lamar St., Dallas, Tex.)

DAVIS, WILSON LORENZO (Assoc. M. '42), Associate Engr. (Civ.), U.S. Engr. Office, Wright Bldg. (Res., 3520 Twelfth Ave.), Sacramento, Calif.

DONNAN, WILLIAM WALTER (Jun. '42), Asst. Civ. Engr., SCS, U.S. Dept. of Agriculture, 117 Eighth St., Imperial (Res., 1226 Olive, El Centro), Calif.

DRUMWRIGHT, HENRY ELMO (Jun. '41), Dist. Engr., State Dept. of Health, Box 268, Tyler, Tex.

ENGLISH, GLENN HARDING (Jun. '41), Junior Engr. (Civ.), Humble Oil & Refining Co., Humble Bldg., Houston, Tex.

ERICKSON, WILLIAM FRANK (Jun. '41), 2d Lt., Corps of Engr., U.S. Army, Asst. Area Engr., Edgewood Arsenal, Md.

EVANS, WILLIAM SPEARING (Jun. '42), 32 High St., New Haven, Conn.

FLETCHER, ADAM GORDON (M. '42), Chf. Civ. Engr., Victorian Rys., Spencer St., Melbourne, Victoria, Australia.

FOSTER, LEWIS EDWARD, JR. (Jun. '41), Junior Engr., U.S. Bureau of Reclamation, 442 U.S. Customhouse, Denver, Colo.

FRANTZ, LUTHER MARSHALL (Assoc. M. '42), Civil Engr., Whitman, Requardt & Smith, 1304 St. Paul St. (Res., 2814 Oakley Ave., Baltimore, Md.

FREY, GEORGE JOHN, JR. (Jun. '41), 2d Lt., Field Artillery, U.S. Army, 12th Battalion, F.A.R.C., Fort Bragg, N.C.

HARRIS, ROBERT SHELDON (Jun. '41), Asst. Engr., U.S. Bureau of Reclamation, 723 North 2d St. (Res., 900 West Roma), Albuquerque, N.Mex.

HAYBROOK, STEPHEN HOWARD (Jun. '41), Junior Engr., U.S. Engrs., Air Depot, Rome, N.Y.

HENDRIX, HUBERT LEE (Assoc. M. '42), Care, du Pont Const. Co., 503 North K St., Muskogee, Okla.

HESTER, QUINCY ADAMS, JR. (Assoc. M. '42), Senior Road Designer, State Dept. of Highways (Res., 533 Spain St.), Baton Rouge, La.

HICKS, ELMER ROBERT (Assoc. M. '42), Tower Engr., Blaw-Knox Co., Blawnox (Res., Delaware Drive, Shaler Park, Glenshaw), Pa.

HIXON, DANIEL ALEXANDER (Jun. '41), Junior Engr., U.S. Engrs., A.O.W., Childersburg (Res., 110 South Broadway, Sylacauga), Ala.

HOLCOMB, JOHN KENNETH (Jun. '41), Ensign, U.S.N., 1753 North 74th St., Wauwatosa, Wis.

HOMBURG, ALBERT HENRY, JR. (Jun. '41), 2d Lt., Corps of Engrs., U.S. Army, Company B, 7th Eng. Training Battalion, Engr. Replacement Training Center, Fort Belvoir, Va. (Res., 3006 Clifton Park Terrace, Baltimore, Md.)

JACOBSON, GEORGE LLOYD (Jun. '41), Care, Civ. Eng. Dept., Univ. of Missouri, Columbia, Mo.

JASKAR, ADE EUGENE (Jun. '42), Asst. Geologist, U.S. Engr. Dept., Univ. of Washington Campus (Res., 111 Lynn St.), Seattle, Wash.

JOHNSON, ROBERT WARD (Jun. '41), 2d Lt., U.S. Army, 425th Coast Artillery Battlion (Comp.), Anti-Aircraft, Army Post Office, 1023 Care Postmaster, New York, N.Y.

KEYSOR, CARL ELLSWORTH (Jun. '41), Asst. Div. Engr., New York Central System, Springfield (Res., Lakeview), Ohio.

KIMMONS, GEORGE HARVEY (Jun. '41), Senior Eng. Aide, TVA, Box 142, Jefferson City, Tenn. (Res., 908 South 2d St., Oxford, Miss.)